BAL GANJOO

Upper Llagas Creek Flood Control Project: Llagas Creek – Stable Channel Design and Sediment Impact Assessment
Final Report

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UPPER LLAGAS CREEK FLOOD CONTROL PROJECT: LLAGAS CREEK – STABLE CHANNEL DESIGN AND SEDIMENT IMPACT ASSESSMENT

1.0 INTRODUCTION

The Upper Llagas Creek Flood Control Project consists of channel improvements and the construction of a newly proposed diversion channel that can provide a 100-year level of protection to urban areas and maintaining the existing levels of protection in the agricultural areas between Morgan Hill and Gilroy in Santa Clara County, California. In addition, the channel improvements also ensure preservation of the existing riparian habitats in various reaches as well as the well-designed stable low-flow channels for vegetation planting. The project area is located in the vicinity of the communities of Morgan Hill, San Martin and Gilroy in Southern Santa Clara County, California, as shown in **Figure 1-1**.

1.1 Study Authorization

The U.S. Department of Agriculture (USDA) under the Watershed Protection and Flood Prevention Act, Public Law 83-566 originally authorized the project in 1969. The Natural Resource Conservation Service (NRCS), previously known as the Soil Conservation Service (SCS), was the public agency to design and construct the Llagas Creek Flood Control Project, under this Public Law (PL) 566. The NRCS began flood control improvement work in 1973. To date, only the lower reaches of the Llagas Creek Flood Control Project (below Buena Vista Avenue) have been completed.

Since 1996, the construction of the upper reaches remained inactive due to lack of funding from the PL-566 program. The Water Resources and Development Act (WRDA) of 1999 authorized the Corps of Engineers to complete the remaining reaches of the project in accordance with the NRCS watershed plan.

1.2 Previous Studies

Several recent studies related to this Upper Llagas Creek Flood Control Project were reviewed and pertinent data were used in the current hydraulic analysis. These study reports are:

1. USACE-SFD (2002). "Upper Llagas Creek Flood Control Project, Sediment Assessment and Hydraulic Design", Draft Report prepared by U.S. Army Corps of Engineers, San Francisco District, December 2002.

2. Abdallah D. Saah (2003). "Llagas Creek Flood Control Project Hydrologic Investigation", Prepared for the Corps of Engineers, San Francisco District.

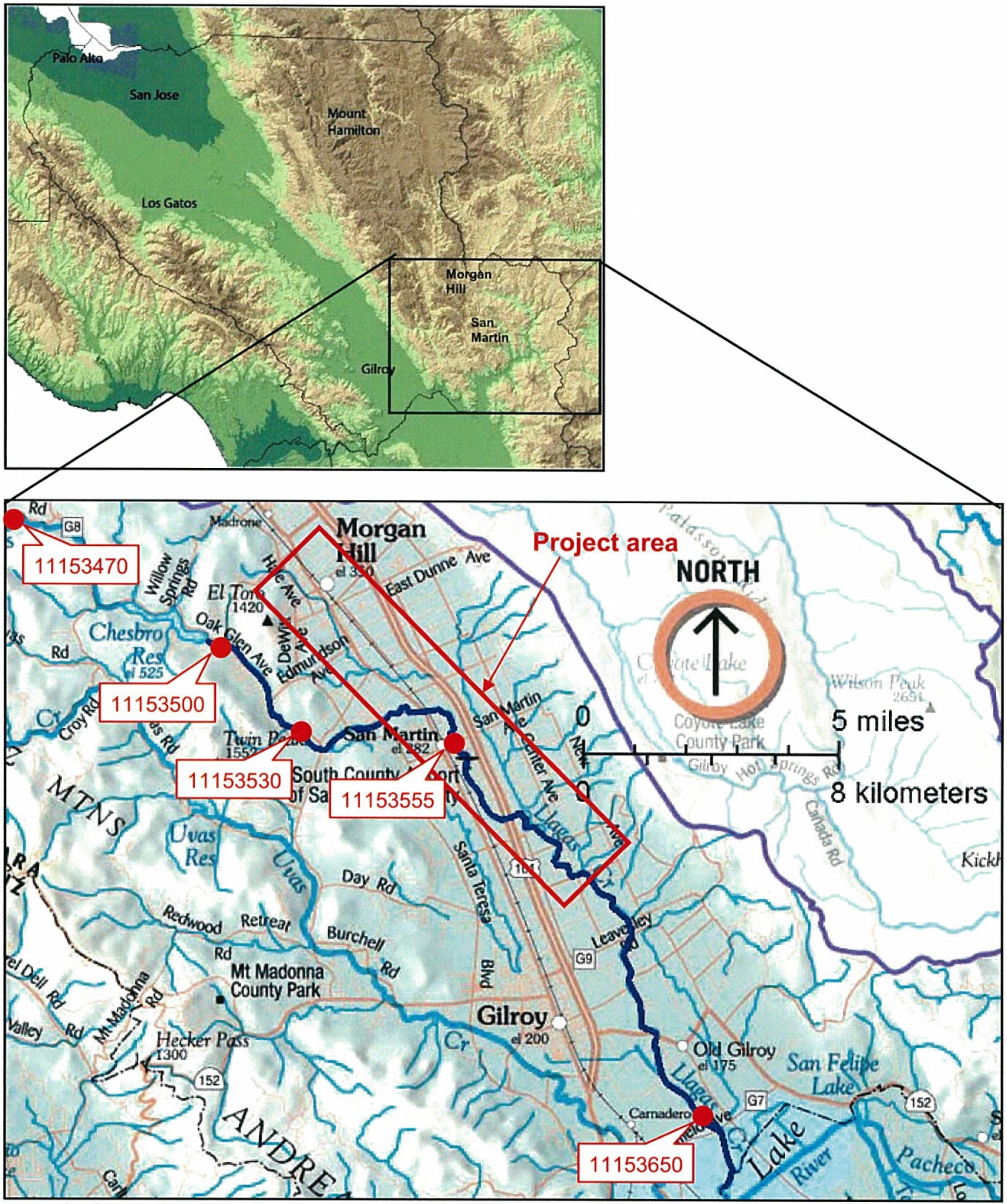
3. USACE-SFD (2006). "Upper Llagas Creek Flood Control Project, Existing Conditions Floodplains Study". Final Report prepared by U.S. Army Corps of Engineers, San Francisco, California, March 2006.

Copy of Report ?

1.3 Purpose and Scope

The objective of this hydraulic study for the Upper Llagas Creek Flood Control project is to provide a reach-specific sediment assessment in the project area, based on the effective discharge for fluvial sediment transport. The results of this study will be used to support a final design and to determine the potential impacts of the proposed flood control project on the sediment transport. A sediment assessment is also needed to aid in the planning and design of the stable channels for the Upper Llagas Creek, as only a general sediment assessment was performed and completed in 2002 (USACE-SFD 2002).

The scoped task requires a detailed hydraulic and sediment analysis to determine the reach specific stable low-flow channel discharge and consequently to size the low-flow channel of a compound channel configuration that was preliminarily formulated during the 2002 study (USACE-SFD, 2002). The potential aggradation and/or deposition problems within the study reaches, including the proposed diversion channel, are to be assessed via the SAM hydraulic program.



Notes: background maps are from http://ggweather.com/climate/ and http://www.pajarowatershed.com

Figure 1-1. Project Location

2.0 GEOMORPHRIC CHARACTERISTICS

2.1 Geographic Setting

The project area, as illustrated in Figure 1-1, is located in the vicinity of the communities of Morgan Hill, San Martin, and Gilroy in Southern Santa Clara County, California. The principal drainage in the project area is Llagas Creek with a drainage system including Llagas Creek, West Little Llagas Creek, and East Little Llagas Creek. Llagas Creek flows in a southeasterly direction to the Pajaro River and ultimately to Monterey Bay and the Pacific Ocean. Tributaries enter Llagas Creek from the Santa Cruz Mountains in the west and the Diablo Range to the east.

The entire upper Llagas Creek watershed covers approximately a 104 square mile area and extends from the upper heights of Mount Loma Prieta in the Santa Cruz Mountains to the lower agricultural plains of south Santa Clara County (USACE-SFD, 2002 & 2006). The watershed is approximately 15 miles long and 6 miles wide, and is bisected along its length by Highway 101. The upper 19 square miles of the watershed are regulated by Chesbro Reservoir, which was built in 1955 as a multipurpose facility for flood control and water supply. The Chesbro Reservoir impounds a maximum conservation pool of 7,500 acre-feet and a maximum pool of 8,086 acre-feet.

The project reach of the Upper Llagas Creek that extends for approximately 14 miles long is located in the middle section of the Llagas Creek watershed and encompasses a drainage area of 61.7 square miles on both the west and east sides of the South Santa Clara Valley. The project boundary extends from a location upstream of Wright Avenue on West Little Llagas Creek and just upstream of Monterey Road on Llagas Creek down to Buena Vista Avenue along Llagas Creek. The west side of the area includes the upper reaches of Llagas Creek, Edmundson Creek, and West Little Llagas Creek. The east side of the area includes the Madrone Channel, Tennant Creek, Foothill Creek, Corralitos Creek, San Martin Creek, Center Creek, New Creek, Church Creek, Rucker Creek, Skillet Creek, East Little Llagas Creek, and the lower reach of Llagas Creek. A schematic of the Llagas Creek watershed is shown in **Figure 2-1**.

2.2 Geology

The geology of the Llagas Creek watershed is typical of the Central Coast Range of California. The watershed can be structurally divided into three sections: the mountainous upper portion, the foothill section, and the lower alluvial valley section (SCVWD, 2001 and USACE-SFD, 2002).

Mountains

The mountainous portion includes the Santa Cruz Mountains and the Diablo Range, running along the western and eastern boundaries of the watershed, respectively. These mountains are characterized by rugged terrain with steep slopes (generally exceeding 15%) and deep valleys. Elevations range from about 500 feet to 1,600 feet in the Diablo Range and to over 3,500 feet in the Santa Cruz Mountains. The Franciscan sedimentary deposits and igneous rocks underlining the mountainous areas are traversed by many large faults that have shattered the adjacent rocks. Erosion processes in the upland regions include the downslope mass movement of soil masses and fractured bedrock, and deepening of the canyons by various streams.

Foothills

The foothill areas have moderate slopes, generally less than 20%. This region is underlain principally by the older alluvial fan deposits of the Quaternary period, and has the well developed soil profiles. The alluvial deposits consist of the consolidated gravels and sand, with the interbedded silt and clay. The thickness of the alluvial deposits is several hundred feet. The older alluvial fans are moderately to deeply weathered, making the considerable amounts of easily erodible material available. These deposits occur in the areas of the slight to moderately sloping topography and are relatively stable. In the areas that have been altered by human activities, gullying and streambank failure are prevalent. Where the slopes have been artificially steepened by construction activities, rotational slumps and other hillslope failures are common.

Alluvial valley

The valley area ranges in elevation from 140 to 400 feet above the Mean Sea Level, with the slopes generally less than 5%. The valley areas are underlain by the thick unconsolidated alluvium that was deposited in the Holocene age. The valley floor is largely coarse-grained material although some limited areas are composed of silt and clay of the floodplain and lacustrine origin. The soils are generally classified as a silty loam of the Post-Aromas sediments. The landscape of this region has been significantly altered by human activities: first primarily by agricultural and more recently by the urban land development. Much of this area has been filled for the development or drained for farming.

2.3 Seismicity

Santa Clara County is located in a seismically active region. The major fault zones transecting Santa Clara County include the San Andreas Fault and Calaveras Fault. The San Andreas Fault is located near the west edge of the county in the Santa Cruz Mountains. The Calaveras Fault bisects the county along the northwest-southeast trend through the Diablo Range. Other faults include the Sargent Fault and Zayante Fault. A sketch showing fault lines in the vicinity of the project area is shown in **Figure 2-2**. These faults have been the source of several large historic earthquakes that have subjected Santa Clara County to strong shaking and are considered the sources of future large earthquakes. Along the San Andreas Fault, a magnitude 8+ earthquake is possible with the associated horizontal displacement of a few tens of feet. An earthquake of the magnitude exceeding 7 is possible along the Calaveras Fault with the lateral displacements of several feet.

2.4 Land Use

Land use in the basin includes the urbanized areas in the upper and lower parts of the watershed, and the agricultural land uses in the mid-section, between the cities of Morgan Hill and Gilroy. In the past thirty years, Santa Clara County has experienced a rapid urban growth; however, the southern part of the county, which includes the Llagas Creek basin, has lagged behind the growth pattern in the northern county.

Nearly one-half of the watershed population lives on 500 separate farm units, most of which are owner operated (USACE-SFD, 2002). About 50% of these farms are in the floodplain area. The land is predominantly devoted to prune orchards with small areas for strawberries and other row crops, grains, and hay. Many walnut orchards are located along Llagas Creek below Chesbro Dam. Poultry and cattle grazing are quite extensive along the valley fringes. Despite an accelerating urban development, agriculture is expected to remain the dominant land use in the Llagas Creek basin.

2.5 Historical Channel Changes

Historical changes in the channel planforms and longitudinal profiles were analyzed using the following topographic data:

- The USGS 1:24,000 quadrangle maps titled Gilroy, Morgan Hill, and Mt. Madonna (USGS 2005). These quad maps are based on the survey conducted in 1955 and are photo-revised in 1981, 1980, and 1994, respectively.
- The USGS 1:62,500 topographic map of Morgan Hill quadrangle surveyed in 1915 and published in 1917 (USGS 2005).
- The cross-section data from the HEC-RAS model developed by the U.S. Army Corps of Engineers (USACE). The HEC-RAS channel geometry is based on the surveys of Llagas Creek, West Little Llagas Creek, and East Little Llagas Creek conducted by Bestor Engineers, Inc. in 2002 and by the San Francisco District of USACE in 2004.

The USGS topographic maps vertical datum is based on the National Geodetic Vertical Datum of 1929 (NGVD29). The Bestor 2002 and USACE 2004 surveys are based on the North American Vertical Datum of 1988 (NAVD88). Therefore, all elevations in the USGS maps were converted to NAVD88 by adding 2.8 feet (USACE-SFD, 2002 & 2006).

Historical channel planform

The planform shift map, as shown in **Figure 2-3,** illustrates changes in the position of Llagas Creek, West Little Llagas Creek, and East Little Llagas Creek over the period from 1915 to 1980-1994. The historical topographic maps were re-projected using a common coordinate system and aligned in GIS. The maximum error in aligning the maps is estimated to be ± 50 feet. Figure 2-3 indicates that the most recent years (1980, 1981, and 1994) represent the years from which the channel alignment on the 1955 USGS quad maps was photo-revised.

The most significant changes in the Llagas Creek planform occurred immediately downstream of Monterey Road, between West San Martin Avenue and Masten Avenue, and downstream of Buena Vista Avenue. The 1915 mapping shows two branches downstream of Monterey Road, while the 1981 mapping shows only a single channel coinciding with the former south branch in this reach. Between West San Martin Avenue and Masten Avenue, the channel of Llagas Creek shifted (mostly westerly) by up to 200-400 feet. Downstream of Buena Vista Avenue, creek was straightened and relocated by up to 300-400 feet. In other reaches of Llagas Creek, channel shifts are less significant and are generally less than 200-250 feet.

Significant planform changes were also observed along the entire project reaches of West Little Llagas Creek and East Little Llagas Creek. The channel of West Little Llagas Creek has been relocated south-westerly by up to 200-400 feet between Llagas Road and Cosmo Avenue and by up to 300-550 feet between Tennant Avenue and Watsonville Road. East Little Llagas Creek has been significantly straightened, resulting in a channel relocation between East Middle Avenue and the creek mouth by up to 300-900 feet. The mouth of East Little Llagas Creek was relocated upstream along Llagas Creek by approximately 4,500 feet and is now located upstream of Masten Avenue.

The observed planform changes were mostly artificial and caused by human intervention, rather than the natural channel evolution processes.

Historical longitudinal profiles

Figure 2-4 and **Figure 2-5** show the existing and historical longitudinal profiles that were respectively developed for Llagas Creek, West Little Llagas Creek, and East Little Llagas Creek. The existing profiles, representing the channel inverts, are based on the Bestor 2002 and USACE 2004 surveys. Historical profiles that were obtained from the USGS topographic maps represent only an approximate mean bed elevation rather than the actual channel invert. The topographic map profiles are therefore usually higher, perhaps by several feet, than the channel invert surveys.

A comparison of the existing and historical profiles of Llagas Creek indicates approximately 5-7 feet bed degradation in the project reach from 1915 to 1955. Further lowering of up to 4-6 feet of the Llagas Creek channel occurred between 1955 and 2002, mostly downstream of East San Martin Avenue.

Less significant changes in bed elevation took place in West Little Llagas Creek and East Little Llagas Creek. From 1917 to 1955, the channel of West Little Llagas Creek incised by up to 3-4 feet downstream of West Dunne Avenue, while the channel of East Little Llagas Creek was relatively stable. From 1955 to 2002, West Little Llagas Creek incised further by up to 5-6 feet and East Little Llagas Creek incised by up to 4-7 feet.

The observed bed degradation most likely resulted from channelization of the creeks and artificial deepening of the creek channels for the flood control purposes, as well as confinement of the creeks by various bridge crossings. The trapping of coarse sediment in Chesbro Reservoir, which was built in 1955, could also contribute to the observed incision of Llagas Creek downstream of the reservoir. Given that the 1955 stream profiles are taken from the topographic maps, whereas 2002 and 2004 profile data are from the channel invert surveys, the estimated channel incisions between 1955 and 2002 are likely to be too high.

The existing profile of Llagas Creek in the project reach has a general concave shape, with the average slopes of approximately 0.0018 (0.18%) immediately upstream of Lake Silveria, 0.0038 (0.38%) between Lake Silveria and Llagas Avenue, 0.0031 (0.31%) between Llagas Avenue and Highway 101, and 0.0022 (0.22%) between Highway 101 and Buena Vista Avenue. The existing West Little Llagas Creek profile has also a concave shape, with the average bed slope of about 0.0024 (0.24%) between Wright Avenue and Watsonville Road. On the contrary, the longitudinal profile of East Little Llagas Creek has a general convex shape, with the average bed slope of about 0.00055 (0.055%) between Watsonville Road and South Pacific Railroad, 0.0026 (0.26%) between South Pacific Railroad and Sycamore Avenue, 0.0052 (0.52%) between Sycamore Avenue and East San Martin Avenue, and 0.0032 (0.32%) downstream of East San Martin Avenue.

The reaches with the abrupt decline in the longitudinal bed slopes where sediment deposition may potentially occur include:

- Llagas Creek immediately downstream of Highway 101,
- West Little Llagas Creek immediately upstream of Watsonville Road,
- East Little Llagas Creek between Watsonville Road and South Pacific Railroad, and
- East Little Llagas Creek immediately upstream of a drop structure at Church Avenue.

The reaches with the increased longitudinal slopes where the streambed erosion may potentially occur include:

- Llagas Creek in the vicinity of South Pacific Railroad crossing,
- West Little Llagas Creek immediately downstream of Wright Avenue,
- East Little Llagas Creek between South Pacific Railroad and Highway 101, and
- East Little Llagas Creek between Sycamore Avenue and East San Martin Avenue.

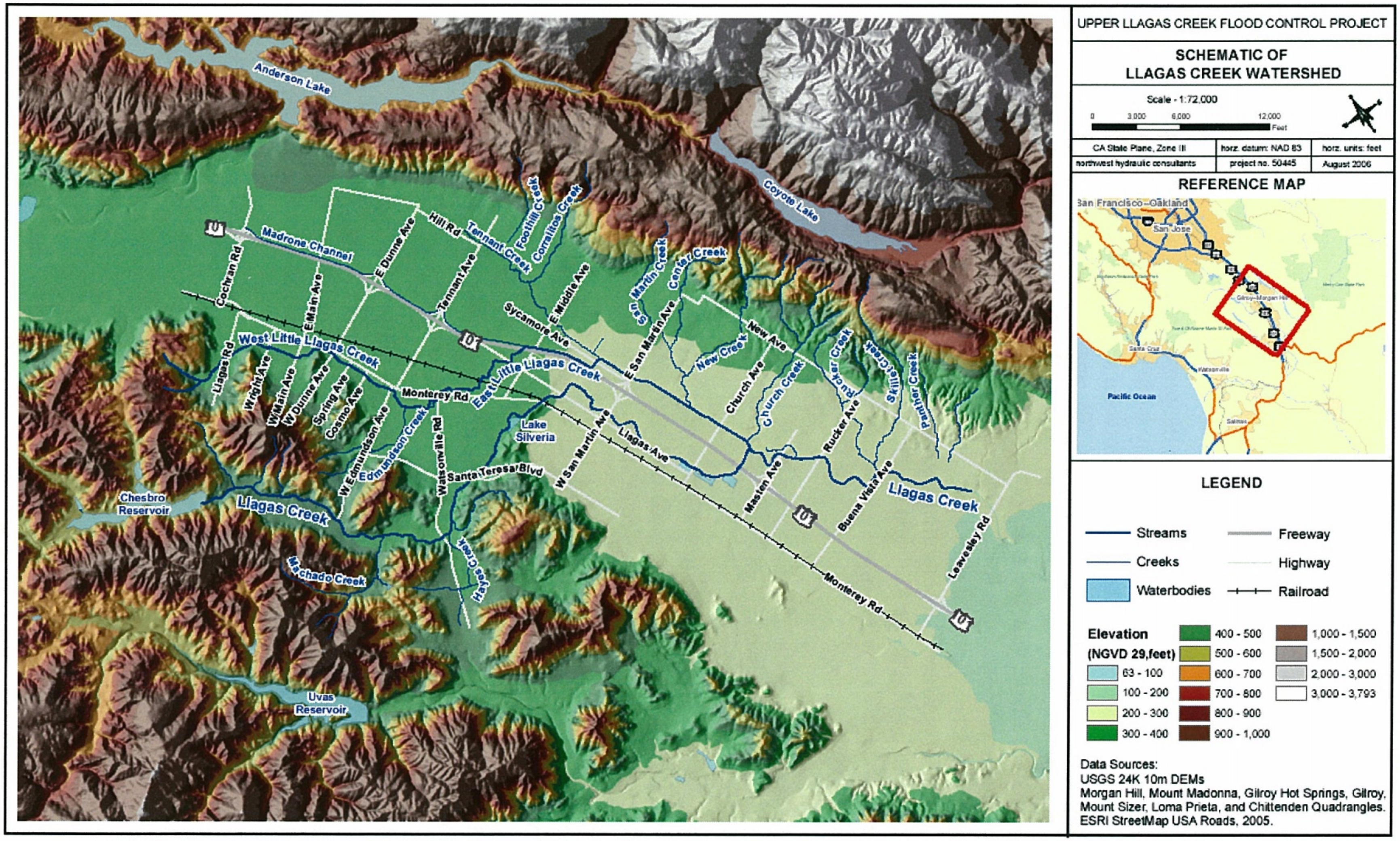
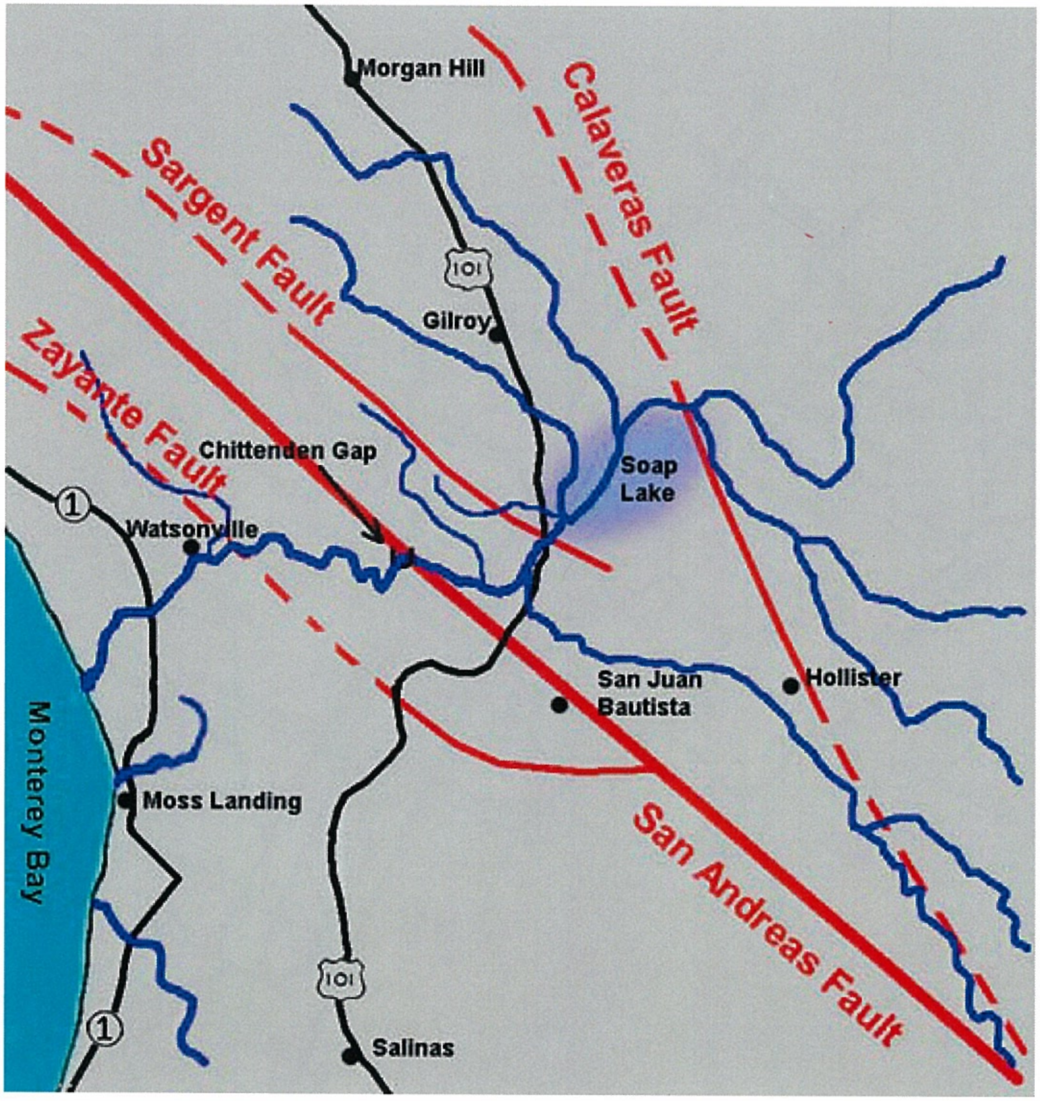


Figure 2-1. Llagas Creek Watershed



Source: http://www.pajarowatershed.com

Figure 2-2. Fault Lines in Vicinity of Project Area

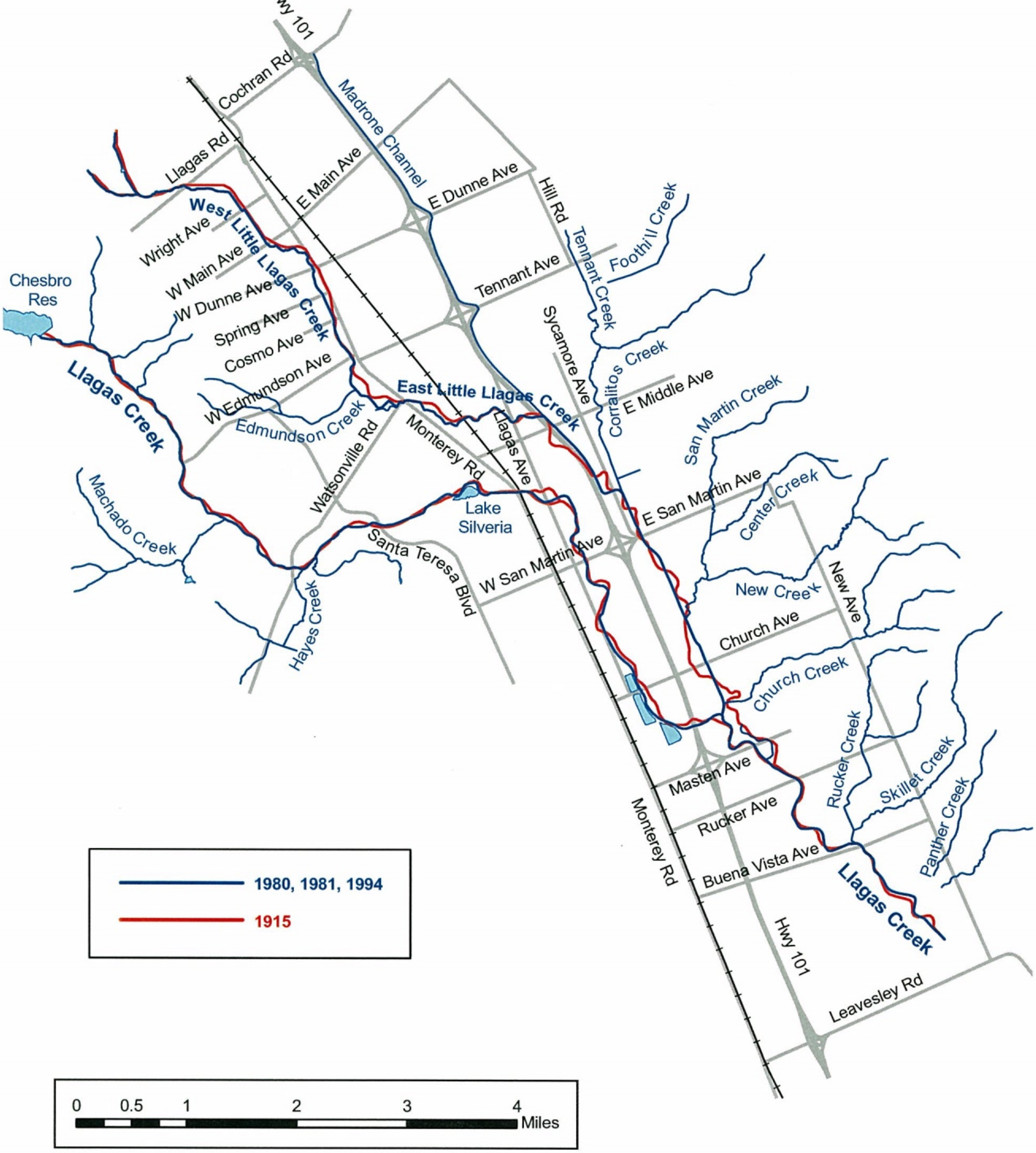


Figure 2-3. Planform Shift Map

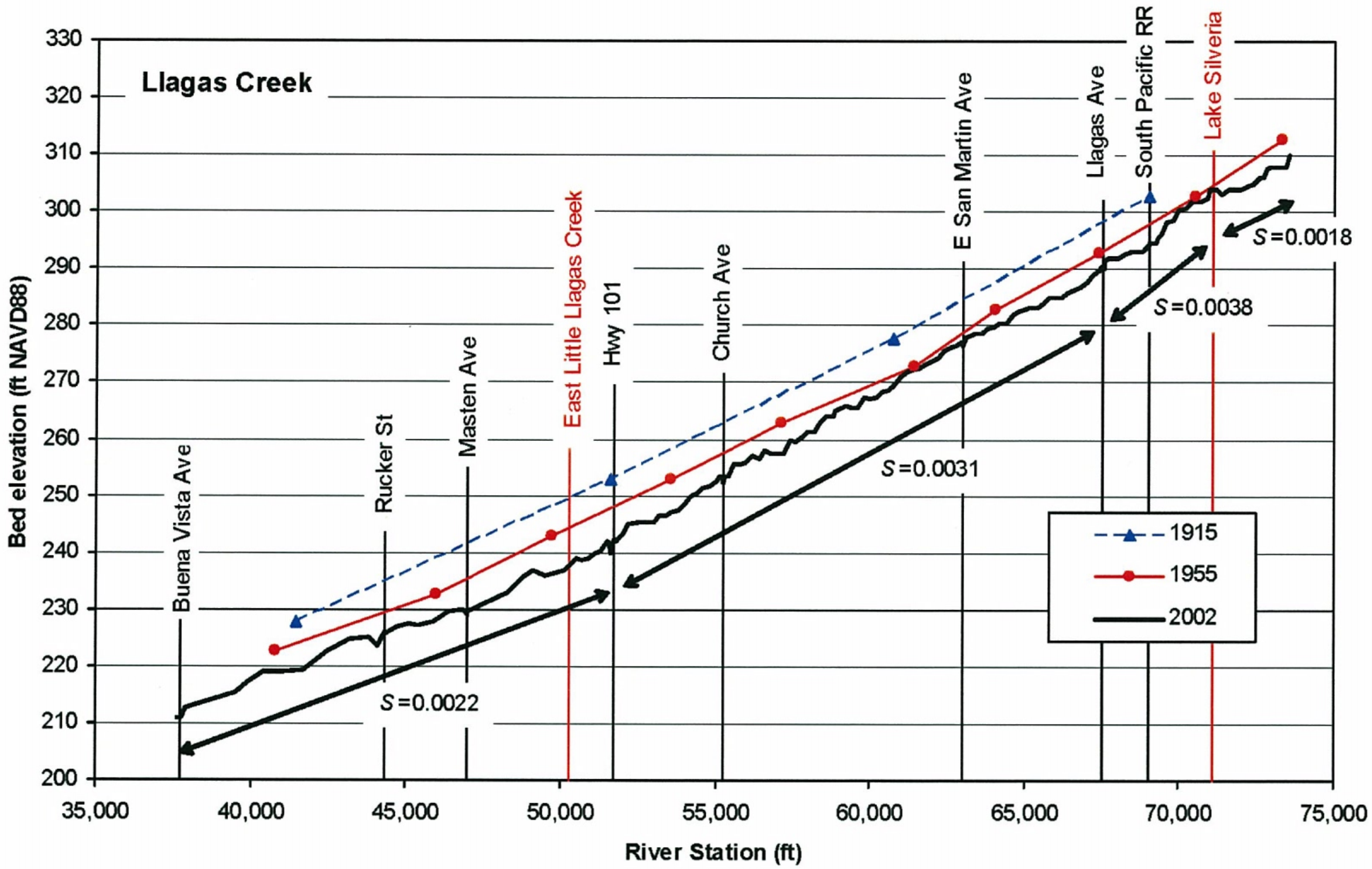


Figure 2-4. Longitudinal Profiles of Llagas Creek.

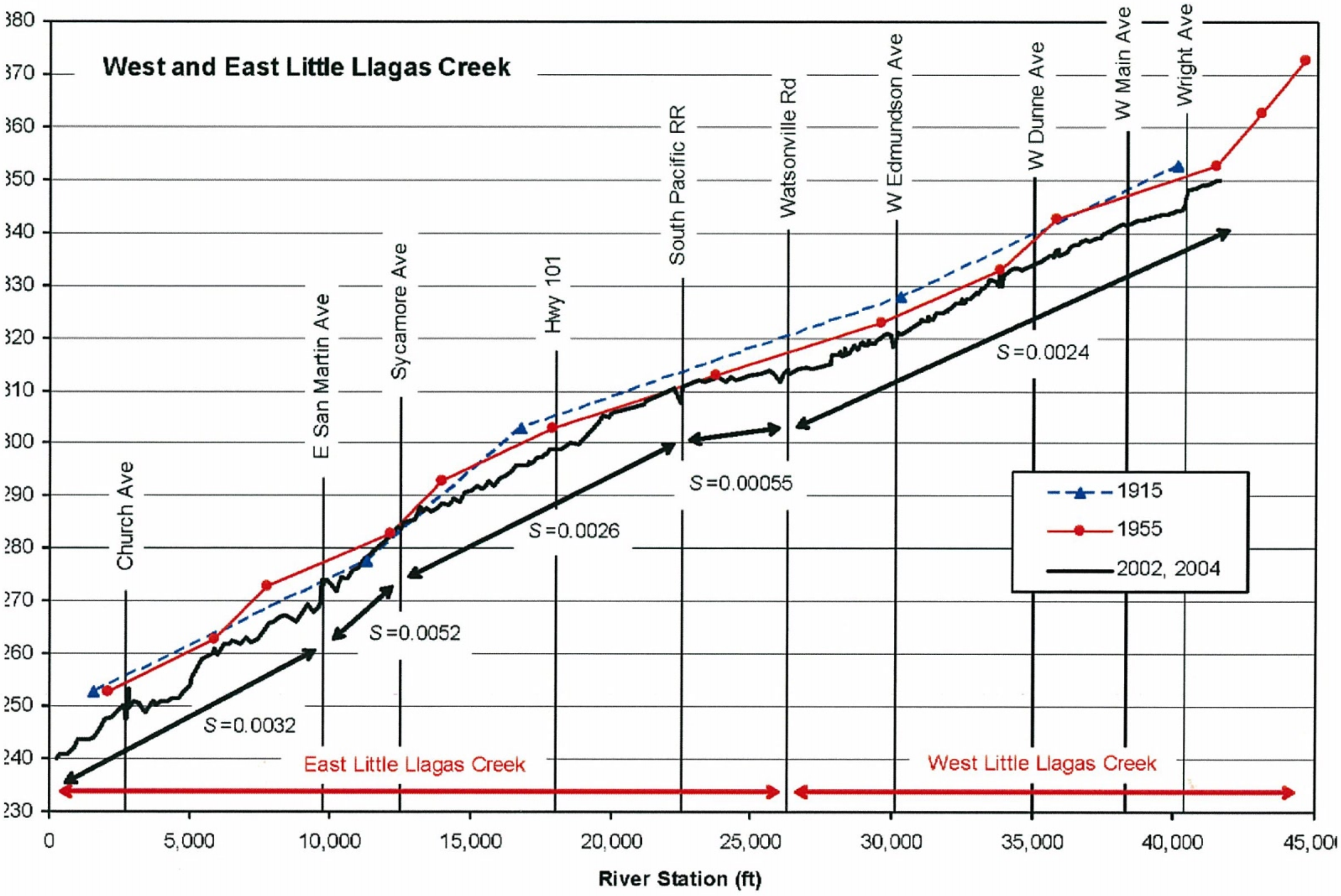


Figure 2-5. Longitudinal Profiles of West/East Little Llagas Creek.

3.0 HYDROLOGIC AND SEDIMENT CHARACTERISTICS

3.1 Regional Climate

The climate of the Upper Llagas Creek watershed is generally moderate with dry, hot summers and mild winters. Based on the meteorological data collected at Gilroy during the 1971-2000 period (NCDC, 2004), the mean monthly temperatures range from 49-53°F in winter to 69-72°F in summer, with the average annual temperature of about 61°F. Extreme temperatures have varied between a low of 17 ° F and a high of 115 ° F.

The rainy season typically lasts from October through May, with the majority of precipitation occurring from December to March. The mean annual precipitation ranges between 18 inches and 46 inches for the west portion of the watershed and is approximately 18 inches for the east side of the study watershed (USACE-SFD, 2006). Snowfall is rare and has no measurable influence on the flood runoff. Flooding within the watershed is almost exclusively associated with the cyclonic storms (USACE-SFD, 2002). These storms, moving in from the North Pacific Ocean, typically have a duration exceeding 12 hours.

3.2 Flow Gage Measurements

The available streamflow records are limited and do not cover all the streams within the project area. Five gages, as shown in Figure 1-1, have been operated by the U.S. Geological Survey (USGS) in the vicinity of the project area, all located in Llagas Creek. The recorded information of these stream gages are listed in **Table 3-1**, including the locations, drainage areas, measured data, periods of records, and counts of measurements. Among the five gages, daily flow discharges were collected only at three gages:

- Gage 11153470 (above Chesbro Reservoir) during 1971-1982 and during 2004-2005,
- Gage 11153500 (below Chesbro Reservoir near Morgan Hill) during 1951-1971, and
- Gage 11153650 (Llagas Creek near Gilroy) during 2002-2006 (low flow record only).

It is noted that USGS Gages 11153470 and 11153500 are the only two stream gages in the project area that have the relatively long data record of the mean daily flow discharge. **Figure 3-1** shows the hydrographs of the mean daily flow measured at Gage 11153500, which is located in Llagas Creek below Chesbro Reservoir near Morgan Hill during the water years of 1962, 1963, and 1969. A water year (WY) is defined as the 12-month period from October 1 to September 30, designated by the calendar year in which it ends. The flow regime of Llagas Creek is characterized by a winter high flow season and a summer low water period. High discharges that originate with rainstorms are typically observed during the winter months when the majority of precipitation occurs. Flooding in Llagas Creek has a "flashy" character, typically lasting for only a few days.

Table 3-1. USGS Stream Gages in Llagas Creek.

Gage No.	Location	Drainage Area (mi^2)	Data Availability
11153470	Above Chesbro Reservoir near Morgan Hill Latitude 37°08'54", Longitude 121°46'02"	9.63	Daily flow: 1971-1982, 2004-2005, 4749 counts Peak flow: 1971-1982, 2004, 12 counts Water-quality samples: 1971-1978, 51 counts
11153500	Below Chesbro Reservoir near Morgan Hill Latitude 37°06'52", Longitude 121°41'22"	19.6	Daily flow: 1951-1971, 7337 counts Peak stream flow: 1952-1970, 20 counts Water-quality samples: 1979-1991, 141 counts
11153530	At Machado School near Morgan Hill Latitude 37°05'23", Longitude 121°39'38"	24.1	Water-quality samples: 1979-1990, 120 counts
11153555	At San Martin Latitude 37°05'13", Longitude 121°36'15"	28.2	Water-quality samples: 1979-1991, 116 counts
11153650	Near Gilroy Latitude 36°59'15" Longitude 121°31'34"	84.2	Daily flow: 2002-2006, 1301 counts (low flow record only, <200 cfs)

Figure 3-2 shows the time series of the annual peak flows and mean annual flows measured at Gages 11153470 (Llagas Creek above Chesbro Reservoir near Morgan Hill) and 11153500 (Llagas Creek below Chesbro Reservoir near Morgan Hill). The construction of Chesbro Reservoir in 1955 generally reduced the peak flows below the reservoir. Above the reservoir, the annual peak flow in Llagas Creek ranged from 8.6 cubic feet per second (cfs) (WY 1977) to 980 cfs (WY 1982), and was 520 cfs on average. Below the reservoir, the annual peak flow ranged from 16 cfs (WY 1960) to 3,190 cfs (WY 1958), and was 525 cfs on average (the average value was obtained for the period after the construction of Chesbro Reservoir). The mean annual flow above the reservoir ranged from 0.2 cfs (WY 1977) to 22.3 cfs (WY 1982), and was 9.7 cfs on average. The mean annual flow below the reservoir ranged from 1.0 cfs (WY 1961) to 33.4 cfs (WY 1958), and was 16 cfs on average. Both the annual peak and mean annual flow series follow a similar temporal pattern. The correlation coefficient between the two data series is 0.94 above the reservoir and 0.78 below the reservoir. This is indicative of the significant contribution of flood flows to the annual runoff in Llagas Creek.

Figure 3-3 shows the major hydraulic characteristics measured at Gage 11153470 (Llagas Creek above Chesbro Reservoir near Morgan Hill) and Gage 11153650 (Llagas Creek near Gilroy). The measured flow ranges from less than 1 cfs to 177 cfs, the stream width from 2 feet to 31 feet, the mean depth (calculated from cross-section area and width) from 0.1 feet to 2.9 feet, and the mean stream velocity from 0.1 to 3.5 feet per second (ft/sec).

3.3 Project Hydrologic Conditions

The hydrology under the with-project condition was characterized by the Corps during the preliminary hydraulic design (USACE-SFD, 2002) and the existing floodplain study (USACE-SFD, 2006). The six study reaches initially defined for the Upper Llagas Creek region were further divided into 16 sub-reaches. The definition of the sub-reaches, the locations, and drainage areas as well as the corresponding River Station (RS) used in the HEC-RAS analysis, is summarized in **Table 3-2**. The reach schematic under the with-project conditions is shown in **Figure 3-4**, as previously defined in the preliminary study (USACE-SFD, 2002).

Table 3-2. Hydrology Summary of 16 Sub-Reaches

Reach	Sub-reach	Location	Drainage area (sq-miles)	River Station (RS)	
8	R8s4	u/s Wright Ave.	1.53	211+00	
	R8s3	u/s main Ave.	1.99	200+30	1070
	R8s2	u/s W. 5th Ave	2.35	177+00	2330
	R8s1	u/s Ciolino Ave	2.61	153+00	2400
7	R7s4	d/s Ciolino Ave	2.61	153+00	
	R7s3	d/s Dewitt Creek	3.37	137+00	
	R7s2	u/s Edmundson Creek	4.09	98+00	
	R7s1	u/s Llagas Creek	5.73	76+00	
6	R6s2	d/s WLLC diversion channel	33.24	718+00	
	R6s1	u/s East Little Llagas Creek	35.5	589+84.25	
5	R5	u/s East Little Llagas Creek	35.5	629+00	
4	R4	d/s East Little Llagas Creek	61.71	382+00	
14	R14s4	u/s Corralitos Creek	6.58	180+00	
	R14s3	u/s San martin Creek	14.54	122+00	
	R14s2	u/s Church Creek	18.96	50+00	
	R14s1	u/s Llagas Creek	20.97	6+00	

The return peak flow rates at different sub-reach locations along the major streams under the with-project conditions were estimated by the Corps (USACE-SFD, 2002 & USACE-SFD, 2006) and are presented in **Table 3-3.** The flow rates for the existing (without-project) conditions were presented in the 2006 report (USACE-SFD, 2006). The estimated 100-year peak flow in Llagas Creek increases from 3,916 cfs downstream of Chesbro Reservoir to 12,568 cfs upstream of Panther Creek. Along West Little Llagas Creek, the estimated 100-year peak flow increases from 636 cfs upstream of Wright Avenue to 2,093 cfs at the proposed diversion to Llagas Creek. Along East Little

Llagas Creek, the estimated 100-year peak flow increases from 2,155 cfs upstream of Corralitos Creek to 5,755 cfs at the confluence with Llagas Creek.

Table 3-3. Flood Event Peak Flow Rates With-Project Conditions

Reach	Sub-reach	Flood Event Peak Flow Rate (cfs) for Various Return Intervals in years							
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	250-yr	500-yr
8	R8s4	186	300	398	450	580	636	720	770
	R8s3	270	430	535	670	750	840	950	1,080
	R8s2	315	500	630	700	880	990	1,120	1,250
	R8s1	344	590	717	930	1,050	1,129	1,300	1,350
7	R7s4	344	590	717	930	1,050	1,129	1,300	1,350
	R7s3	454	780	944	1,200	1,400	1,483	1,675	1,700
	R7s2	542	840	1,050	1,300	1,500	1,579	1,775	1,850
	R7s1	673	1,100	1,357	1,700	1,900	2,093	2,350	2,450
6	R6s2	1,101	2,400	3,276	4,000	4,700	5,532	6,400	7,000
	R6s1	1,101	2,400	3,276	4,000	4,700	5,532	6,400	7,000
5	R5	1,101	2,400	3,276	4,000	4,700	5,532	6,400	7,000
4	R4	2,840	5,400	7,122	9,600	11,100	12,568	14,500	15,900
14	R14s4	929	1,225	1,563	1,800	2,000	2,155	2,400	2,500
	R14s3	1,187	2,150	2,535	3,200	3,600	4,051	4,550	4,900
	R14s2	1,419	2,500	3,149	3,800	4,400	5,136	6,000	6,800
	R14s1	1,490	2,750	3,450	4,100	4,850	5,755	6,900	7,900

Source: USACE-SFD, 2006

3.4 Sedimentation

Limited sediment data were collected by USGS at four water-quality sampling stations in Llagas Creek (see Table 3-1). Bed material was sampled above Chesbro Reservoir at Gage 11153470 in 1972 and 1977 (altogether 9 samples), below the reservoir at Gage 11153500 in 1985 (2 samples), at Machado School near Morgan at Gage 11153530 during the 1985-1987 period (4 samples), and at San Martin at Gage 11153555 in 1985 and 1986 (altogether 3 samples). Discrete suspended load and bed load samples were collected only above Chesbro Reservoir at Gage 11153470. In total, 41 suspended load samples were collected during the 1971-1978 period and only one bed load measurement was conducted in 1978. Sediment sampling at all the four stations has been discontinued since 1987.

Figure 3-5 shows the cumulative grading curves of bed material in Llagas Creek that were derived from the available USGS data (USGS 2006). The median bed material sizes ranged from 1.90 mm to 20.1 mm at Gage 11153470 (above the reservoir), from 0.47 mm to 0.54 mm at Gage 11153500 (below the reservoir), from 0.78 mm to 1.04 mm at Gage 11153530 (at Machado School), and from 0.68 mm to 0.83 mm at Gage 11153555 (at San Martin). The average median sizes of bed material at these stations were 7.18 mm, 0.49 mm, 0.88 mm, and 0.73 mm, respectively. The bed material in Llagas Creek upstream of the reservoir was composed mostly of gravel. The bed material at the gages located below the reservoir was mostly medium to coarse sand.

Figure 3-6 shows the cumulative grading curves of the suspended sediment and the single bed load sampled in Llagas Creek above Chesbro Reservoir. The suspended sediments were composed of silt and sand, while the bed load was mostly composed of medium to coarse sand and fine gravel. The median size of the suspended sediment ranged from 0.006 mm to 0.18 mm with an average value of 0.015mm. The median size of the bed load was 0.82 mm. The size of the suspended sediment appears to be independent of the magnitude of creek flow. Considerable scatter of the median sediment data is quite typical for fluvial sediments, which reflects the natural variability of the sediment transport processes in rivers.

Figure 3-7 shows the relationship between the measured suspended load discharge and stream flow. The measured bed load discharge (only one data point) is also shown in this figure. For the flow rates between 9 cfs and 850 cfs, the suspended load discharge varied from 1 ton/day to 10,800 tons/day, with the concentration of the suspended sediment varying from 12 mg/L to 4,710 mg/L. Both the suspended and bed load were measured in Llagas Creek above the reservoir on February 9, 1978. On this day, the flow in the creek was measured to be 134 cfs, the measured suspended load discharge was 44 tons/day (about 98% of the total sediment load) and the measured bed load discharge was 0.82 tons/day (about 2% of the total sediment load). The relationship between the suspended load discharge and flow rate shown in Figure 3-7 can be approximated by the following equation:

$$Q_s = 0.000516 \times Q^{2.44}$$

Where Q_s the suspended load is discharge (tons/day) and Q is the flow (cfs). The correlation coefficient for this relationship is 0.96

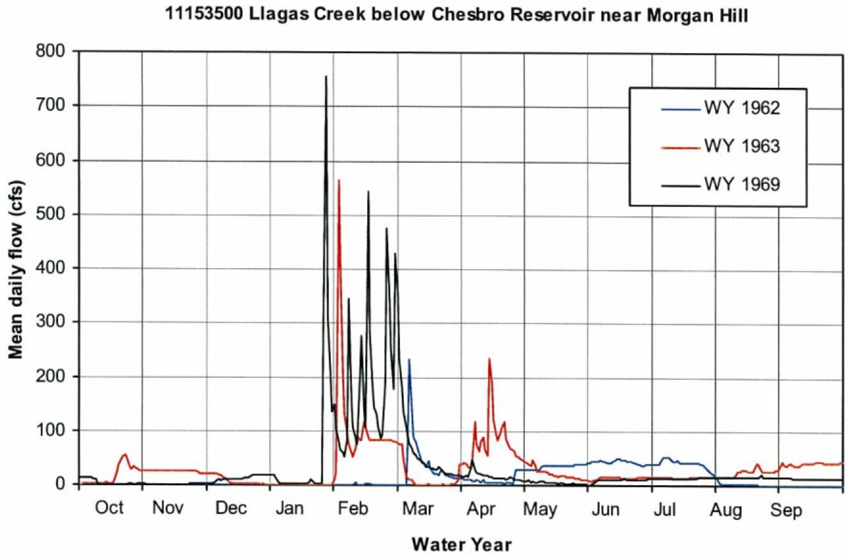


Figure 3-1. Mean Daily Flow Hydrographs at Gage 11153500 in Years 1962, 1963 & 1969

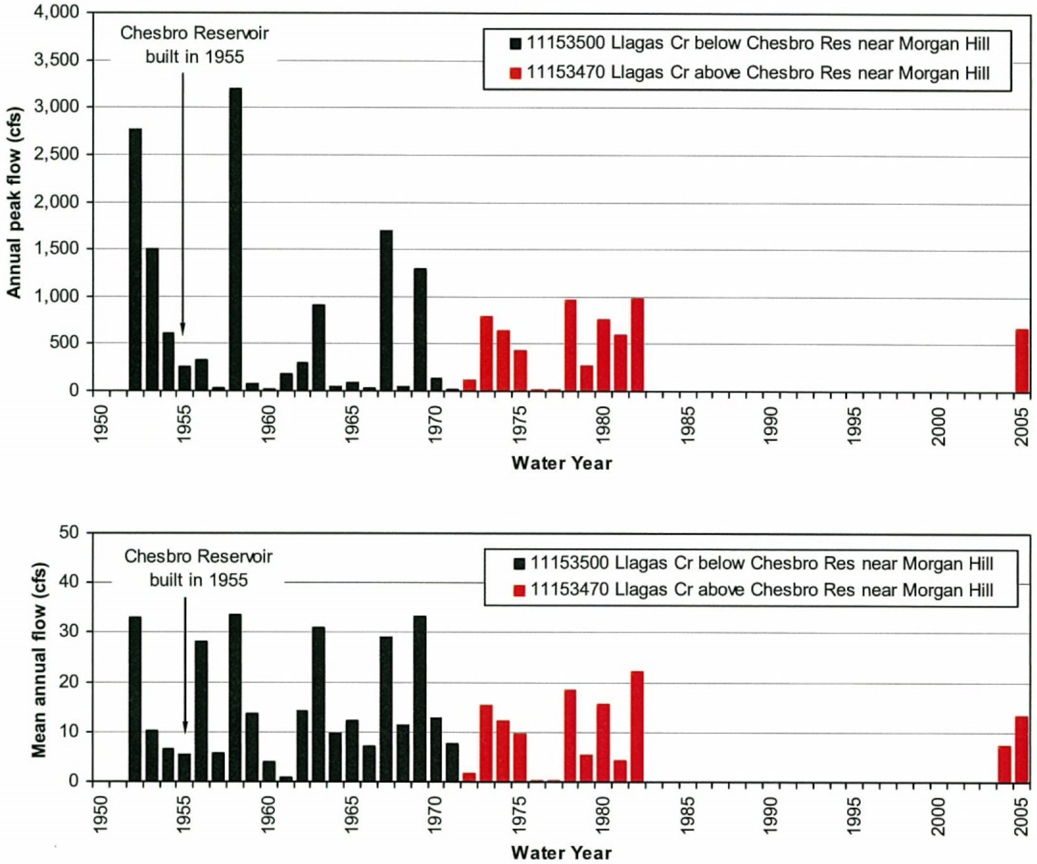


Figure 3-2. Time Series of Annual Peak Flow (top) and Mean Annual Flow (bottom)

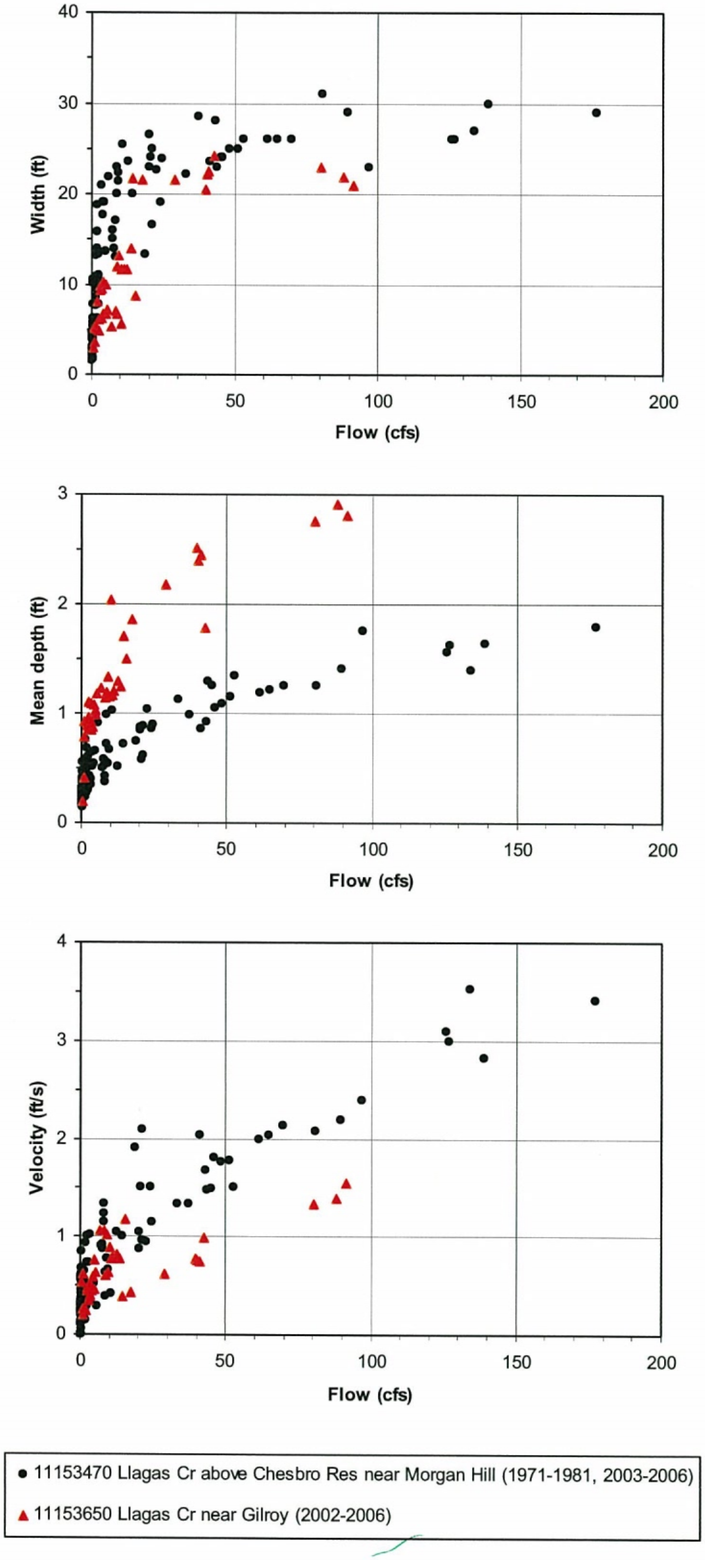
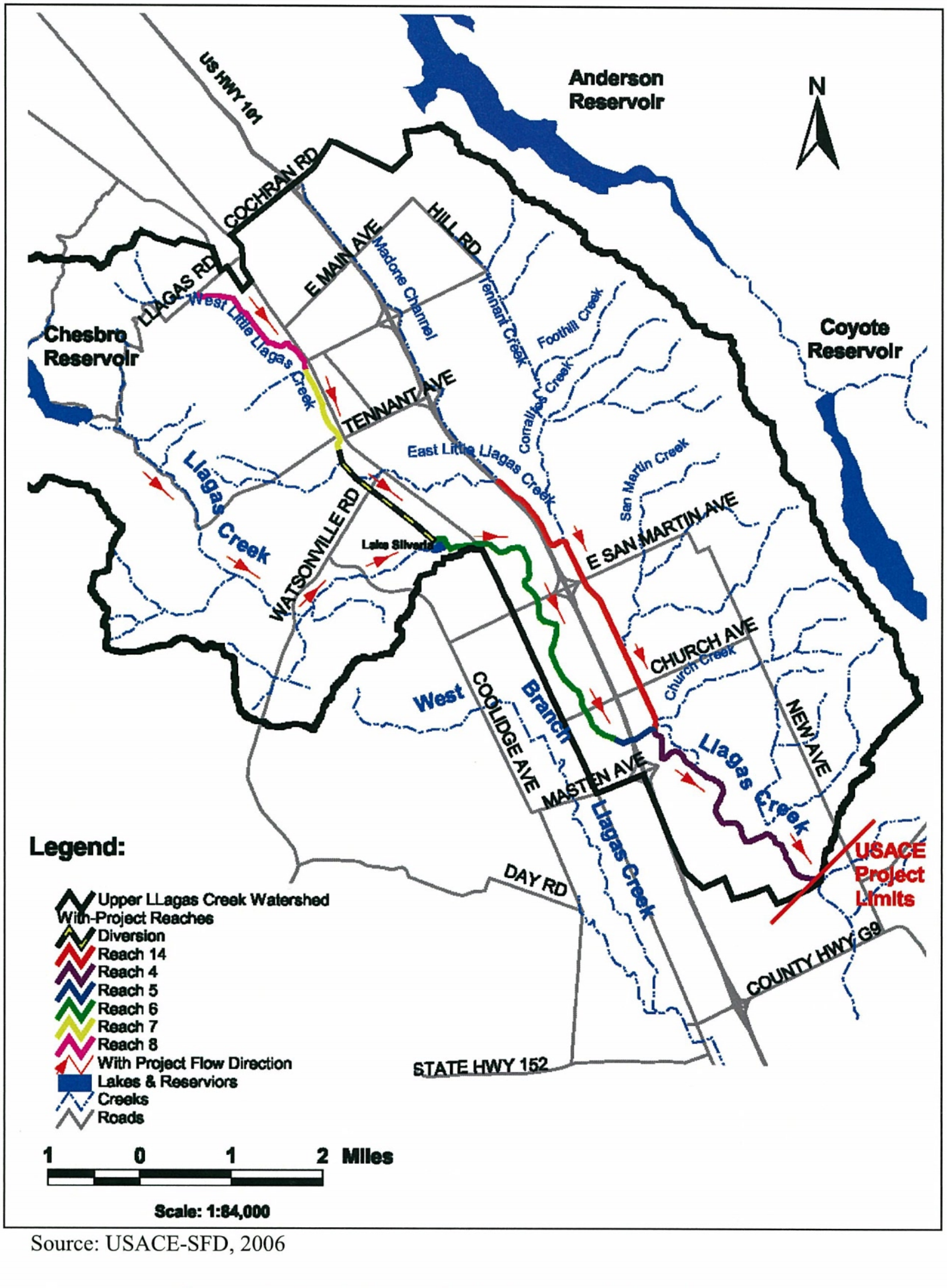


Figure 3-3. Measured Hydraulic Characteristics of Llagas Creek



Source: USACE-SFD, 2006

Figure 3-4. Reach Schematic Under With-Project Conditions

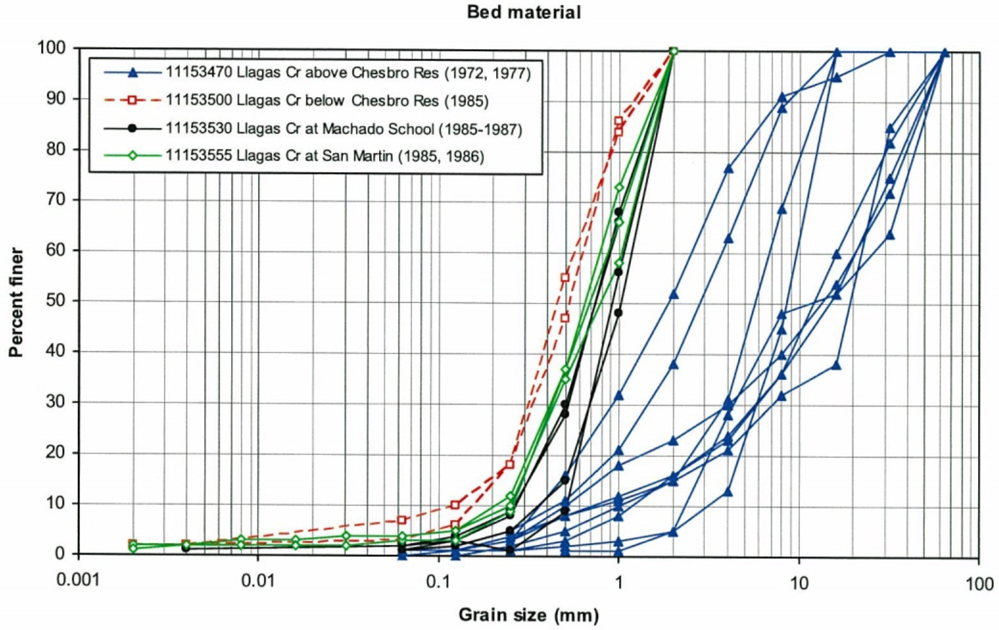
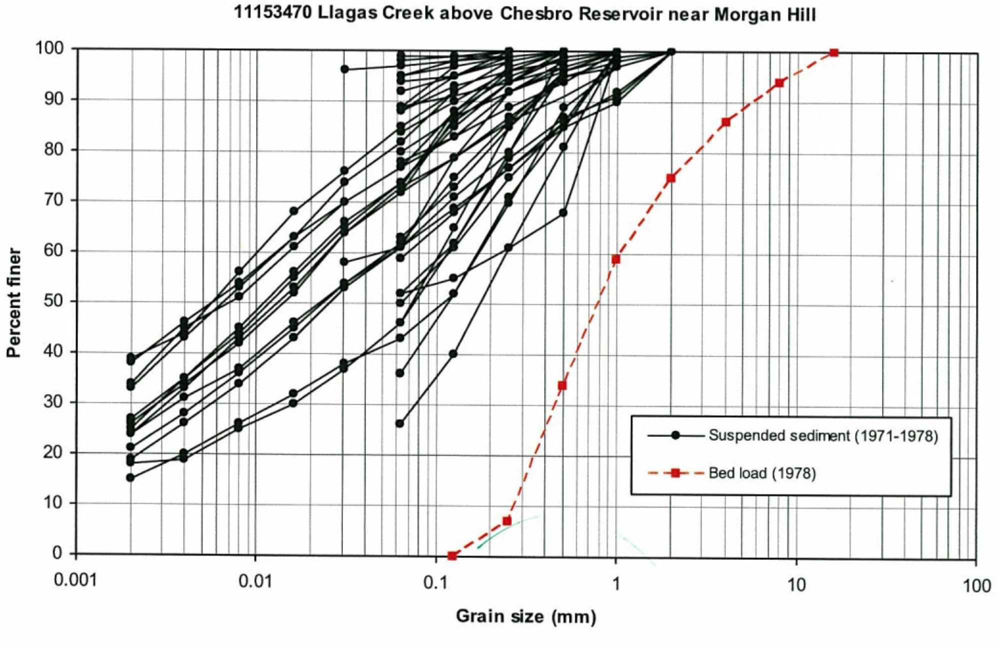


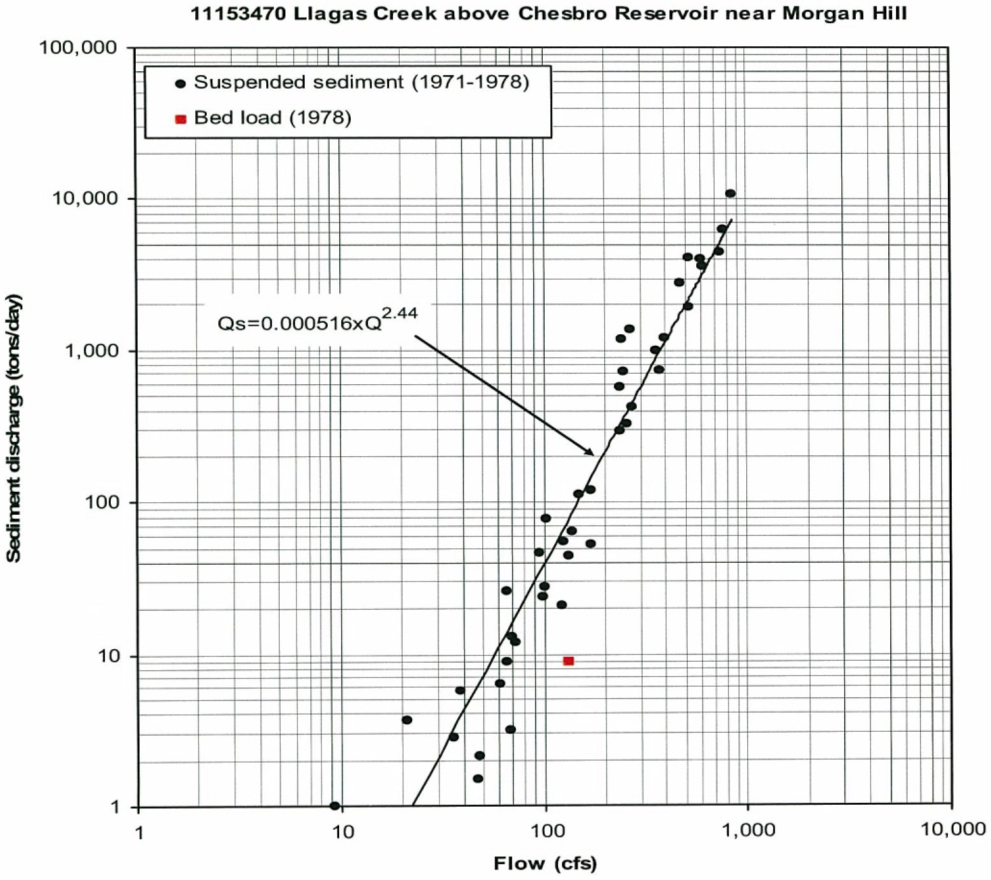
Figure 3-5. Cumulative Grading Curves of Bed Material in Llagas Creek

Not consistent with Photo 56



Note: at USGS gage 11153470

Figure 3-6. Cumulative Grading Curves of Suspended Sediment and Bed Load in Llagas Creek



Note: at USGS gage 11153470

Figure 3-7. Sediment Discharge versus Flow

4.0 FIELD RECONNAISSANCE

A field reconnaissance of the major streams within and in the vicinity of the project area was conducted on July 23-24, 2006. The inspected reaches included:

- West Little Llagas Creek from Llagas Road to Watsonville Road,
- East Little Llagas Creek from Watsonville Road to Church Avenue, and
- Llagas Creek from Chesbro Reservoir to Leavesley Road.

Ground photographs from this field reconnaissance were taken as: Photos 1-28 for various reaches of West Little Llagas Creek: Photos 29-44 in East Little Llagas Creek; and Photos 45-77 in Llagas Creek. These photos are shown in Appendix A. Following is a general description of the geomorphic features that were observed in the study reaches of these three creeks.

4.1 West Little Llagas Creek

West Little Llagas Creek (WLLC) is mostly located within an urbanized area consisting of high-density residences, single-family residences, and large commercial developments. Several homes and buildings have been built next to the channel of the creek (see Photo 13).

The study reach of the creek is characterized by an artificially constructed, trapezoidal earthen channel, with a top width averaging about 20 feet and an average depth of 4 feet (Photos 7, 8, 13, 15, 17, 21, 23, and 27). The stream flow in the creek is typically seasonal, except in a few locations where the perennial water occurs due to the urban runoff.

Four different geomorphic reaches can be identified within the study reach of West Little Llagas Creek:

WLLC-1 - Upstream of Hillwood Lane,
WLLC-2 - Between Hillwood Lane and West Edmundson Avenue,
WLLC-3 - Between West Edmundson Avenue and about 1,000 feet downstream of South La Crosse Drive, and
WLLC-4 - Further downstream to Watsonville Road.

In Reach WLLC-1 and Reach WLLC-3, the creek mostly flows through a compound, two-stage channel consisting of a low flow channel and a overbank flow section (Photos 1, 2, 23, and 24). In Reach WLLC-2 and Reach WLLC-4, the creek flows through a single low flow channel, with the flat overbank areas that are frequently occupied by residential homes and commercial buildings (Photos 8, 13, 15, 17, 18, 20, and 21).

The creek channel consists mostly of various relatively straight sub-reaches, following the alignment of local streets and roads. The creek channel has several abrupt 90-degree turns at Hale Avenue upstream of Hillwood Lane, between West Main Avenue and West Dunne Avenue, and between South La Crosse Drive and Watsonville Road. An incomplete section of the proposed diversion channel joins West Little Llagas Creek at a location about 1,000 feet downstream of South La Crosse Drive (Photos 25 and 26). Some sections of the channel are concrete-lined and are protected with sacked concrete, particularly in the vicinity of bridges and culverts (Photos 9-11).

Dense in-channel vegetation growing in many reaches of Llagas Creek (Photos 1-3, 6, 12, 21- 24, and 27) can be observed. The in-channel vegetation stabilizes the streambed and traps sediment. Mature trees grow along the creek banks at some locations (Photos 3, 5, 12-13, 17, 20, and 21). Litter is damped in the creek channel at some places along the residential areas (Photos 9 and 17). The bed material in West Little Llagas Creek ranges from muddy deposits to gravel.

A total of 17 bridges and culverts are located in the project reach of WLLC (downstream of Hillwood Lane). At least 3 more culverts are located upstream of the project reach between Hillwood Lane and Llagas Road. Some of the bridges and culverts are shown in Photos 6, 10, 11, 14, 16, 19, 22, and 28. The dimensions and capacity of each bridge and culvert vary, however all appear to be undersized. The bridges and culverts represent the in-channel obstacles affecting the flow and sediment transport in the vicinity of the structures. Localized bank and bed erosion was observed at some of the culverts (Photo 4).

An analysis of the longitudinal profile, as shown in Figure 2-5, identified a reach of the creek immediately downstream of Wright Avenue as a site of the potential channel erosion and a reach upstream of Watsonville Road as a channel area of the potential sediment deposition. A field inspection of the creek downstream of Wright Avenue (see Photo 9) did not reveal an apparent sign of channel erosion, although the bank protection installed below the culvert is indicative of the likely channel stability problems in this reach. The creek reach located upstream of Watsonville Road (Photo 27) appears to be a depositional area, which is evidenced by a dry, vegetated channel, and fine sediment deposits. During the high-flow events, floodwaters overtop Watsonville Road and flow in a southerly direction along the east side of Monterey Road, eventually spilling over Monterey Road into East Little Llagas Creek (USACE-SFD, 2006).

4.2 East Little Llagas Creek

East Little Llagas Creek (ELLC) flows through a mostly agricultural area with the scattered single-family residences and small commercial developments, which are sometimes located next to the creek channel. No flow was observed in the creek during the field reconnaissance. East Little Llagas Creek is typically dry in the summer and fall months (USACE-SFD, 2006).

Two different geomorphic reaches can be identified in East Little Llagas Creek:

ELLC-1 - Upstream of Highway 101, and
ELLC-2 - Downstream of Highway 101.

Upstream of Highway 101, the creek consists of a small earthen channel, mostly covered with dense grass and weeds (Photos 32 and 33). The channel width ranges from 10 to 50 feet and the channel depth ranges from 2 to 8 feet. Mature trees grow along creek banks at some locations (Photos 32 and 35). A total of 10 culverts and bridges cross East Little Llagas Creek in this reach. The capacity of the creek channel, culverts, and bridges upstream of Highway 101 is insufficient to convey high flows, which is evidenced by frequent flooding in this area (USACE-SFD, 2006). During the field reconnaissance, debris deposited probably by recent floods was observed on the top of Watsonville Road and Monterey Road Culverts (Photos 29 and 31). The banks at some culverts are protected with sacked concrete (Photos 29 and 35). Some of the bank protection works show signs of distress (Photo 29).

Downstream of Highway 101, East Little Llagas Creek flows in an excavated trapezoidal channel (Photos 36 and 43). The channel of the creek was straightened and realigned by CalTrans in the 1970's during the construction of Highway 101. The top width of the channel is typically between 50 and 100 feet with the channel depth being within 5-10 feet. This creek reach has a channel capacity of 25-year flood event within most of the reach (USACE-SFD, 2006). In total, 4 culverts cross the creek between Highway 101 and the confluence with Llagas Creek. These culverts are capable of conveying a 10-year flood event (USACE-SFD, 2006). The channel protection measure such as rock revetment is installed at some crossings and channel bends (Photos 36 and 38). A section of the channel at East San Martin Avenue is lined with concrete (Photo 39). Drop structures are constructed downstream of East San Martin Avenue Culvert (Photo 39) and upstream of Church Avenue Culvert (Photo 42). The Channel bed is scoured below some of the drop structures (Photo 39). The bed material of East Little Llagas Creek is mostly gravel (Photos 30, 37, 41, and 44). In many locations downstream of Highway 101, the gravel bed is covered with a layer of dry algae (Photos 37, 40, and 41).

An analysis of the longitudinal profile (see Figure 2-5) identified a reach of East Little Llagas Creek immediately upstream of South Pacific Railroad as a depositional area. This reach is characterized by a very small channel (Photo 33), which does not have the capacity to convey significant flows and large sediment loads. The South Pacific Railroad Culvert (Photo 34) is undersized and elevated, acting as a barrier to the channel flow. Floodwaters can overtop the channel upstream of the railroad, inundate the low-lying area to the south, and eventually spill over Monterey Road into Llagas Creek (USACE-SFD, 2006).

Another reach identified as a depositional area is located upstream of Church Avenue (Photo 42). The drop structure constructed immediately upstream of the Church Avenue culvert acts as a dam, accumulating incoming sediment upstream of the structure.

The reaches with a locally increased longitudinal slope where erosion may potentially occur are located between the South Pacific Railroad and Highway 101, and between Sycamore Avenue and East San Martin Avenue. A field inspection of the creek at Llagas Avenue that is located between South Pacific Railroad and Highway 101 revealed the exposed tree roots in the creek banks and coarse-grained streambed (Photos 35 and 57). These are the typical signs of channel erosion. The channel at Sycamore Avenue Culvert is protected with rock revetment (Photo 38), which is indicative of the channel erosion problems in this reach.

4.3 Llagas Creek

The study reach of Llagas Creek (LC) is mostly located in an agricultural area, with the scattered single-family homes and small commercial developments. Several homes, green houses, and buildings have been built next to the channel banks. The stream flow in Llagas Creek is regulated by the operation of Chesbro Reservoir (Photo 45).

The study reach of Llagas Creek can be divided into three geomorphic reaches:

LC-1 - Between Chesbro Reservoir and Monterey Road,
LC-2 - Between Monterey Road and Highway 101, and
LC-3 - Downstream of Highway 101.

Upstream of Monterey Road, the creek represents a typical steep-gradient mountain stream that is characterized by the step-pool morphology and rapid currents during summer low flow (Photos 46 and 52). During the field inspection, the stream velocity in this reach was about 1-2 feet per second, the water depth was a few feet, and the streamflow width was about 5-10 feet. Dense vegetation (bushes and mature trees) grows along the creek banks (Photos 47-49, and 52). The creek bed is mostly composed of gravel and cobble deposits (Photos 48 and 50) with occasional rock outcrops.

Between Monterey Road and Highway 101, Llagas Creek flows through a better defined low-flow channel with the gentle longitudinal slopes resulting in a decreased stream velocity and an increased water depth (Photos 53, 55, and 57-58). The top width of the creek channel is on the order of 100-200 feet and the total channel depth is around 10-15 feet. The channel is heavily vegetated (Photos 53-54, 55 and 57-58). Bed material on the channel surface is mostly composed of gravel (Photos 56 and 59).

Downstream of Highway 101, Llagas Creek is contained in a 50-150 feet wide and 10-15 feet deep trapezoidal channel (Photos 63, 65, 69, 71, and 74-75). The channel was dry during the field inspection. The channel banks in this reach are composed of gravel and fine sediment (Photos 62 and 73). The bed material is presented by fine to coarse gravel (Photos 66 and 67). The channel bottom is generally flat without the regular bed forms (Photos 68, and 71). In some places, the gravel bed is partially or completely covered with dry algae and grass (Photos 75, and 77). The mid-channel vegetated gravel bars were observed at several locations (Photos 63 and 69). The stream flow is deflected around the bars, which in some cases promote local bed scour (Photo 70). Large, mature trees grow along the channel (Photos 65, 68, 69, 71, 72, and 74). Various exposed tree roots were observed at some channel locations (Photo 72), indicating that the bank is erosive during high flows.

In total, 10 bridges and culverts cross the creek in the project reach, all of which have the capacity to convey the flood flow of a 100-year flood event. At least 8 bridge and road crossings exist between Chesbro Reservoir and the project reach. The channel and bank protection is installed at some bridges and culverts (Photos 64 and 76). Minor streambed scour occurred at the piers of Rucker Avenue Bridge (Photo 67), around the bed protection works at Masten Avenue Bridge (Photo 64), and downstream of Leavesley Road Bridge (Photo 77).

Several lakes are located along Llagas Creek within the study area. Lake Silveria (Photo 51) is situated approximately 1,700 feet upstream of Monterey Road. Lake Silveria was historically a quarry pit. The maximum storage capacity of this lake is approximately 31 acre-feet at elevation of 307 feet, NAVD88. During a storm event that exceeds the 2-year flood condition, the flood water flows from the Llagas Creek weirs into Lake Silveria, thus lowering the peak flow in the creek. Several small ponds (apparently former quarry pits) separated from the creek by the levees in different heights are located in the vicinity of Church Avenue and upstream of Highway 101 (Photos 60 and 61).

An analysis of the changes in the longitudinal profile as shown in Figure 2-4 indicated the potential degradational reach in the vicinity of the South Pacific Railroad crossing and the potential depositional reach immediately downstream of Highway 101. Llagas Creek in these reaches was not inspected during the field reconnaissance due to the restricted access.

4.4 Bed Material Sampling

Bed material sampling was conducted for the purpose of deriving the sediment grain size distributions, which were used to estimate the sediment transport rates for various flow discharges. In total, sixteen soil samples, including fifteen bulk sediment samples and one surface sample, were collected from the creek bed during the field reconnaissance. The bulk sediment samples were typically taken within a half inch below the bed surface. At least one sample was taken within each of the eight previously defined reaches (see Figure 3-4).

The fifteen bulk sediment sampling locations include (1) six locations in West Little Llagas Creek (four located upstream of Dunne Ave, one between Ciolino Drive and Edmunson Creek, and one downstream of Edmunson Creek); (2) four locations in East Little Llagas Creek (one upstream of Highway 101, one between Highway 101 and Tennant Creek, one between Tenant Creek and San Martin Creek, and one between Tenant Creek and the confluence with Llagas Creek; and (3) six sites in Llagas Creek (three upstream, and three downstream of the confluence with East Little Llagas Creek). The one surface sample was taken immediately upstream of the Rucker Avenue crossing in Llagas Creek. The coordinates and locations of each sample are presented in **Table 4-1**, while the site photographs of the soil sampling locations are presented in Appendix B.

Table 4-1. Geographic Coordinates and Locations of Bed Material Sampling

Sample	Coordinates	Location
1	E121° 40.166’, N37° 08.136’	WLL, upstream of Duane Ave.
2	E121° 39.857’, N37° 08.077’	WLL, upstream of Duane Ave.
3	E121° 39.483’, N37° 07.695’	WLL, upstream of Duane Ave.
4	E121° 39.104’, N37° 07.573’	WLL, upstream of Duane Ave.
5	E121° 38.932’, N37° 07.273’	WLL, Ciolino Dr. to Edmundson Cr.
6	E121° 38.574’, N37° 06.515’	WLL, downstream of Edmundson Creek
7	E121° 38.149’, N37° 06.440’	ELL, upstream of HWY 101
8	E121° 35.732’, N37° 05.350’	ELL, Tennant Cr. to San Martin Cr.
9	E121° 36.063’, N37° 05.751’	ELL, HWY 101 to Tennant Cr.
10	E121° 35.055’, N37° 04.308’	ELL, San Martin Cr. to confluence
11	E121° 33.549’, N37° 02.935’	LC, downstream of confluence with ELL
12	E121° 34.216’, N37° 03.469’	LC, downstream of confluence with ELL
12a	E121° 34.216’, N37° 03.469’	LC, downstream of confluence with ELL
13	E121° 35.731’, N37° 04.143’	LC, upstream of confluence with ELL
14	E121° 37.061’, N37° 05.734’	LC, upstream of confluence with ELL
15	N/A	LC, upstream of confluence with ELL

Notes: WLL: West Little Llagas Creek; ELL: East Little Llagas Creek; LC: Llagas Creek.

A sieve analysis was conducted by Fugro West Inc. on all fifteen bulk sediment samples, and the hydrometer analysis was also performed on several samples that have more than 20 percent sediment passing sieve Number 200. The samples that were subjected to the hydrometer analysis include Samples 1, 2, 5, 6, 7 and 9. **Figure 4-1** shows the derived sediment grain size distributions for the fourteen samples that were taken within the study area. The individual plots of the grain size distribution are also included in Appendix B. As shown in Figure 4-1, the particle size of bed material varies significantly within the study area. The bed material in Reach 8 is generally much finer and shows a much larger scattering range than other reaches. The median grain size (D_{50}) ranges from 0.013 millimeters in the upper Reach 8 to as coarse as 24 millimeters in Reaches 6 and 14.

It is worthy to point out that the bulk sediment samples were taken within one-half-foot depth from the bed surface. The sediment grain size derived from the samples well represents the bed material characteristics in the layer immediately below the creek bed surface, which is strongly dependent on the local sedimentation or erosion process. This might be the main reason for the sediment grain size varying significantly within the study reaches. If the depth of bed material sampling is sufficiently deep, the particle size distribution of the bed material is expected to show much less spatial difference. In order to estimate the long-term sediment budget and the resulting creek sedimentation/erosion, the sediment grain size distribution curves derived from the fourteen bulk samples were averaged, and the mean distribution curve, as shown in **Figure 4-2**, was used as the representative sediment grain size distribution for the entire study area.

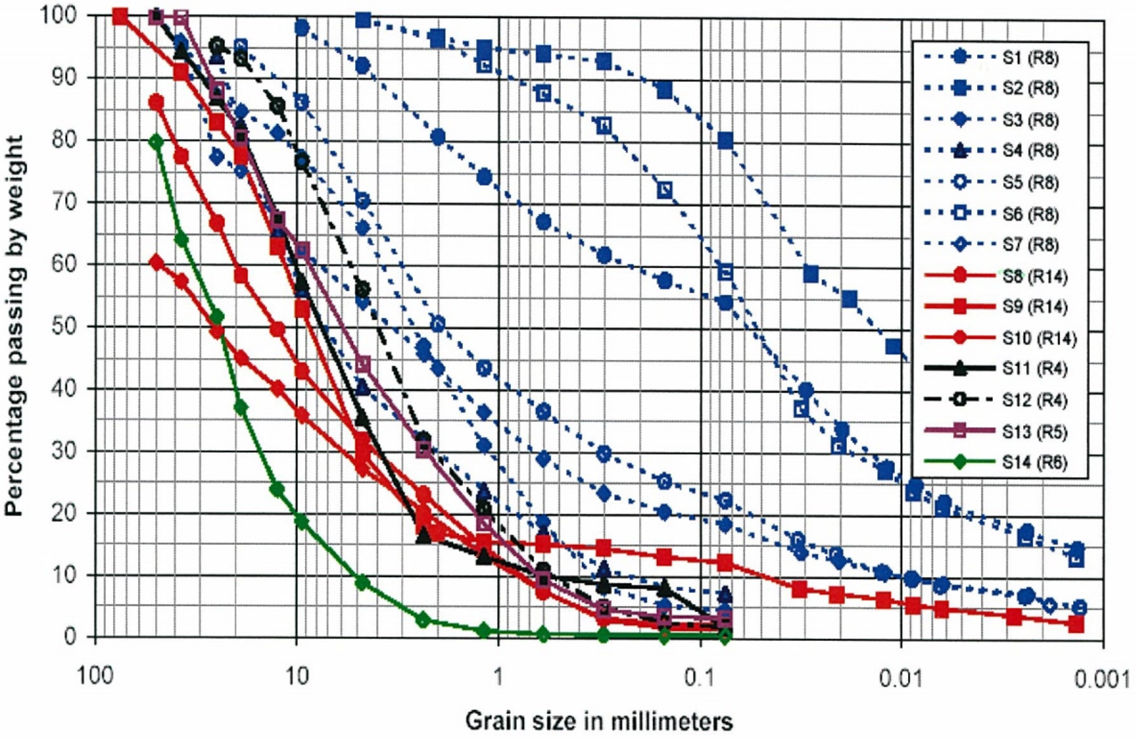


Figure 4-1. Sediment Grain Size Distributions of the Fourteen Bed Material Samples

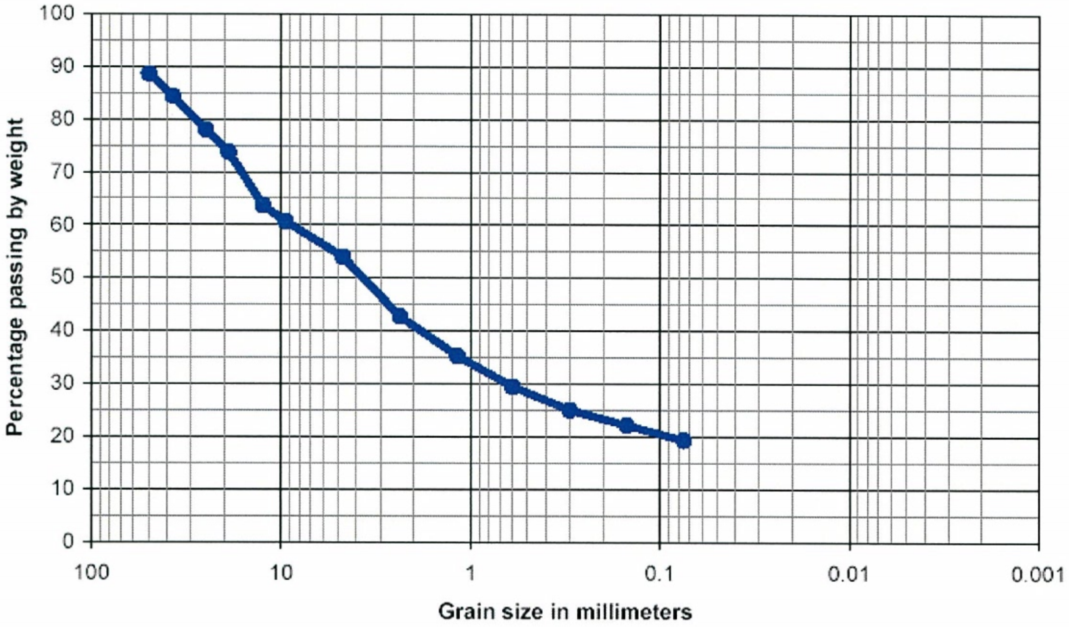


Figure 4-2. Sediment Grain Size Distribution Averaged over the Fourteen Samples

5.0 STABLE CHANNEL DESIGN AND SEDIMENT IMPACT ANALYSIS

The preliminary hydraulic design prepared in 2002 (USACE-SFD, 2002) incorporated the channel modification to provide the flood protection against the 100-year flood event in Reaches 7 and 8, the 10-year event in Reaches 5, 6, and 14, and the 5- to 10- year event in Reach 4. The hydraulic design also included the low-flow channel that was based on the 2-year flow event. In order to modify and further improve the preliminary channel design, several documented methods for the stream restoration projects (Copeland et al., 2001) were evaluated and a pertinent method was selected in this analysis.

5.1 Determination of Stable Low-Flow Channel Discharge

The philosophy of the river hydraulic design is to employ the available physically-based methodologies to determine the design variables of channel width, depth, slope (i.e., the channel longitudinal gradient) and planform for various given independent variables. These variables typically are the water inflow, sediment inflow, bank composition, and, in some cases, the downstream water surface elevation. The four main methodologies that are typically used to formulate the river restoration design include (a) the reference reach technique, (b) the regime equations and hydraulic geometry relationships, (c) the extremal methods, and (d) the dominant discharge approach.

The reference reach technique is to assign the channel geometry similar to an existing stable channel in the project reach or in a reference reach. The project and reference reaches are physiographically similar in the watershed and the composition of channel material, and have no significant hydrologic, hydraulic, or sediment differences. The method of the regime equations and hydraulic geometry relations is to relate a design variable, such as width or slope, to an independent driving variable, such as discharge or drainage. The hydraulic geometry relations can be developed from field observations at the stable and alluvial cross sections in a river or watershed with similar physiographic characteristics. Neither the reference reach technique nor the method of the regime equations and hydraulic geometry relations was appropriate for the hydraulic design of the Llagas Creek hydraulic system, since no stable channel was identified in this creek's hydraulic system, or in a nearby watershed that has similar physiographic characteristics.

If a reliable geometry relationship cannot be determined from the field data, an extremal hypothesis, such as the hypothesis of the minimum energy expenditure rate or the maximum sediment transport potential, may be employed to obtain a range of feasible solutions of channel geometry. However, the extensive field experience demonstrates that channels can be stable with the geometry different from those observed under the extremal conditions. Also the sensitivity of the energy minima or sediment transport maxima to changes of the driving variables may be low that the channel dimensions corresponding to the extremal values are poorly defined; and thus the extremal methods were also not employed in this hydraulic design.

The dominant discharge approach is to obtain a family of solutions in terms of slope (i.e., longitudinal gradient) and depth for different specified channel widths under a selected discharge. These solutions represent combinations of channel width, depth, and slope that satisfy the sediment transport and resistance equations. Once one of the dependent variables is selected based on the established project constraints, the other two can readily be determined. Because of the superior theoretical background of this approach as well as the physical setting and constraints of Llagas Creek, the dominant discharge approach was applied to improve the Llagas Creek hydraulic system that was preliminarily designed in 2002 (USACE-SFD, 2002).

The dominant discharge is a single steady discharge that given enough time would produce the channel dimensions equivalent to those produced by the natural long-term hydrograph. This discharge therefore dominates the channel form and process and is normally used to formulate the morphological inferences. Although conceptually attractive, the definition of the dominant discharge is not necessarily feasible or universally accepted. The following deterministic approximations for the dominant discharge have been suggested (Copeland, et. al, 2001): (1) The natural bank-full channel discharge, which is the maximum discharge that the channel can convey without overflowing onto the floodplain; (2) A discharge based on a statistical return interval of 1 to 2.5-years; and (3) The effective discharge, which is defined as the mean of the discharge increment that transports the largest fraction of the annual sediment load over a period of years. The effective discharge is a function of both the magnitude of a flow event and its frequency of occurrence. An advantage of using the effective discharge is that it is a calculated value not subject to the problems associated with the determination of the field indicators. It is rather calculated by integrating a flow-duration curve and a sediment-rating curve. Therefore, the effective discharge was used to represent the dominant discharge in this stable low-flow channel design.

In accordance with the hydrologic conditions of Llagas Creek, the six study reaches, Reaches 8, 7, 6, 5, 4 and 14 (see Figure 3-4) were further divided into 16 sub-reaches. The stable channel design and sediment impact assessment were then conducted for each of the sub-reaches. The definition of the sub-reaches follows the similar discretization that was used in the preliminary study (USACE-SFD, 2002). The definition of the sub-reaches, the locations, drainage areas, and derived peak flow discharges are presented in Tables 3-2 and 3-3.

5.1.1 Flow-Duration Curves

Determination of the bed material load histogram and the effective discharge is dependent of the derived flow-duration curve. As presented in Table 3-1, five gages have been operated by the U.S. Geological Survey (USGS) in Llagas Creek, and the mean daily flow discharges were collected only at Gages 11153470 (above Chesbro Reservoir) and 11153500 (below Chesbro Reservoir near Morgan Hill). However, neither of these two gages is located in any of the study sub-reaches. Therefore, the regionalized duration curve method for the ungaged site that was recommended by the Hydraulic Design of Stream Restoration Projects (Copeland, et al., 2001) was used for synthesizing the flow-duration curves of all the 16 sub-reaches, based on the flow-duration curves at these two gaged sites.

Figure 5-1 shows the daily mean stream flow data that were collected only at Gage 11153470 during 1971-1982 and during 2004-2005, and at Gage 11153500 during 1951-1971. The derived flow duration curves are shown in **Figure 5-2**. The flow duration curve at Gage 11153470 indicates that the mean daily flow in Llagas Creek above Chesbro Reservoir was greater than 100 cubic feet per second (cfs) during 1.7 percent of the time, greater than 10 cfs during 17.7 percent of the time, and greater than 1 cfs during 45.2 percent of the time. Below the reservoir at Gage 11153500, the mean daily flow was greater than 100 cfs during 2.1 percent of the time, greater than 10 cfs during 34.1 percent of the time, and greater than 1 cfs during 75.7 percent of the time.

In this analysis, the 2-year peak flow discharge was used to proportionally transfer the flow duration relationship from the gage sites to the 16 sub-reaches. For simplicity, the 2-year discharges of the two selected stream gages were estimated by proportionally interpolating the relationship between the drainage area and the 2-year peak flow discharge that was estimated for Reach 14, as listed in Table 3-3. The respective drainage areas are 9.63 square miles for Gage 11153470 and 19.6 square miles for Gage 11153500. The deduced 2-year peak flow discharges are 1,028 cfs for Gage 11153470 and 1,442 cfs for Gage 11153500, respectively.

The regionalized flow duration curve was normalized for each of the two gages by dividing the discharges in the flow duration relationship by the 2-year discharge of the gage site. The derived dimensionless flow-duration curves are shown in **Figure 5-3** for the two gage sites. The average regionalized flow-duration curve was then developed, as also shown in Figure 5-3. The flow duration curve for each of the sixteen sub-reaches was subsequently calculated. Each flow duration curve is the multiplier of the averaged dimensionless discharge index and the 2-year peak flow discharge of each respective sub-reach as listed in Table3-3. The computed flow-duration curves are respectively shown in **Figures 5-4** to **5-19** for the sixteen reaches. As an example, the derived flow duration curve for Reach 4, as shown in Figure 5-15, indicates that the mean daily flow discharge of Reach 4 was greater than 1,000 cfs during 0.14 percent of the time, greater than 100 cfs during 5.6 percent of the time, greater than 10 cfs during 40.2 percent of the time, and greater than 1 cfs during 79.8 percent of the time.

5.1.2 Sediment Transport Rating Curves

The sediment transport rating curve was determined for each sub-reach using the sediment transport functions selected in the SAM program (Thomas, et al., 2002). Four sediment transport functions included in SAM, the Yang function, the Einstein total load function, the Ackers-White function, and the Engelund-Hanson function, were used in this analysis to compute the bed-material sediment discharge for each representative flow discharge. These four empirical functions were specifically selected for the purpose of including the uncertainty range of the calculated sediment discharges. The geometric mean sediment discharge, averaged over the results that were calculated from the four functions, was used as the final sediment transport rating curve, based on which the effective discharge and sediment budget were estimated.

Figures 5-20 to **5-35** show the calculated sediment transport rating curves for the sixteen sub-reaches, respectively. The results computed from the four functions as well as the geometric-mean values are all plotted in these figures. As expected, a wide range of sediment discharge was calculated from the four functions, and the uncertainty range is approximately a factor of one to two. Among the four functions, the Ackers-White function appears to predict the largest sediment discharge for most of the given flow discharges, and the Einstein function yields the minimum sediment discharge. It is also noted that the sediment transport rating curves vary significantly for different sub-reaches as a result of the varying local channel geometry, channel longitudinal profile, and bed material property among the sub-reaches.

For the purpose of a realistic confirmation of the calculated sediment transport rating curve, the collected field data of the instantaneous suspended sediment discharge and water discharge collected at USGS Gage 11153470 were analyzed. Gage 11153470 is located upstream of Chesbro Reservoir and is beyond the study reaches. However, it is the only gage in the vicinity area that has the measured sediment discharge data. In total, 41 suspended load samples were collected at this gage from 1971 to 1978. **Figure 5-36** shows the relationship between the measured suspended load discharge data and stream flow discharge data at this gage site, and the derived curve best fitting the data. Also shown in this figure is the computed mean sediment transport rating curve for Sub-reach R6S2, which is located downstream of the Chesbro Reservoir. The comparison, as shown in Figure 5-36, indicates that the sediment transport rating curve computed for R6S2 is on the approximately same order of magnitude as estimated from the field data collected at USGS Gage 11153470.

5.1.3 Effective Discharge

The effective discharge, used to represent the dominant discharge in this stable low-flow channel design, is defined as the mean of the discharge increment that transports the largest fraction of the annual sediment load over a period of years. The effective discharge incorporates the principle that the channel-forming discharge is a function of both the magnitude of the event and its frequency of occurrence. The flow duration curve and the sediment transport rating curve were integrated to derive a sediment histogram that displaying sediment discharge as a function of the water discharge. The histogram peak is defined as the effective discharge increment. In this histogram analysis, twenty five discrete discharge classes with an arithmetically equal interval were used to cover the full range of water discharge for each sub-reach.

The derived sediment discharge histogram for all the sixteen sub-reaches are shown in **Figures 5-37** to **5-52**, respectively. The discharge increment corresponding to the histogram peak was used to represent the effective discharge of each sub-reach, as presented in **Table 5-1.** For the 16 sub-reaches in the project area, the estimated effective discharges range from 42 cfs for sub-reach R8S4 to 629 cfs for Reach 4. In general, a larger effective discharge was calculated for a reach with a greater flow discharge. It is also noted that the effective discharge, which is expected to be close to the 2-year mean daily flow discharge, is much smaller than the 2-year peak flow discharge that was used in the preliminary low-flow channel design (USACE-SFD, 2002).

Table 5-1. Estimated Effective Discharge for 16 Sub-Reaches

Reach	Sub-reach	Effective discharge (cfs)	2-year peak discharge (cfs)
8	R8s4	42	186
	R8s3	62	270
	R8s2	70	315
	R8s1	78	344
7	R7s4	78	344
	R7s3	101	454
	R7s2	122	542
	R7s1	151	673
6	R6s2	244	1,101
	R6s1	244	1,101
5	R5	244	1,101
4	R4	614	2,765
14	R14s4	208	929
	R14s3	263	1,187
	R14s2	317	1,419
	R14s1	333	1,490

5.2 Stable Low-Flow Channel Design

The stable channel analytical method presented in the Corps hydraulic design package SAM was used to size the stable low-flow channels with the derived reach specific effective discharges. This method is based on a typical trapezoidal cross section for a steady, uniform flow condition. Using the SAM program, a family of solutions in slope (i.e., longitudinal gradient) and depth can be obtained for different specified widths with each derived effective discharge. These solutions represent the combinations of channel width, depth, and slope that satisfy the sediment transport and roughness equations. For a sandy creek, SAM uses the sediment transport and resistance equations that were developed by Brownlie (1981). The wide range of possible solutions can be narrowed by the assigned project constraints.

To the extent possible, the channel design criteria are: (a) a composite channel geometry with a low-flow channel to carry the effective discharge, and (b) within the proposed right of way, the overbank to sustain the 100-year flood in Reaches 7 and 8, and at a minimum to maintain the existing channel conveyance for Reaches 4, 5, 6 and 14. Because of the presence of the control structures such as bridges and culverts in Llagas

Creek, the design bottom slope of the creek remains the same as the preliminary design (USACE-SFD, 2002). The slope constraint was used in this design to size the low-flow channel dimension from the solution curves that were calculated from SAM.

As discussed in the preliminary study report (USACE-SFD, 2002), Reach 8 and the upper portion of Reach 7 are either too narrow to include a low-flow channel or will be replaced with the reinforced concrete culverts. In addition, the channel in Sub-reach R6S2 remains undisturbed in order to preserve the existing local riparian habitats. Therefore, the creek channels formulated in the preliminary design for these sub-reaches will not be modified. The stable channel analytical methods programmed in SAM were only applied to the remaining 10 sub-reaches including R7S3, R7S2, R7S1, R6S1, R5, R4, and all four sub-reaches in Reach 14.

The families of the slope-width and depth-width solutions calculated from SAM for each reach specific effective discharge are shown in **Figures 5-53** to **5-61** for the ten sub-reaches, respectively. For each sub-reach, the base width of the low-flow channel was then determined by interpolating the slope-width solution curve using the channel slope used in the preliminary design. The water depth was determined by interpolating the depth-width curve using the derived base width. The results indicate that the low-flow channel base-width and water-depth solutions can only be determined for five sub-reaches R7S3, R6S1, R5, R14S4 and R14S2. No solution can be derived for the other five sub-reaches (R7S2, R7S1, R4, R14S3 and R14S1) owing to the abrupt steepness or flatness of the longitudinal channel profile in these sub-reaches as compared to the upstream sediment supply reaches.

The sized low-flow channel geometry is listed in **Table 5-2** for the five sub-reaches where a stable low-flow channel can exist. The base width of the low-flow channel ranges from approximately 25 feet in R7S3 to 54 feet in R14S2, and the low-flow channel depth varies between 1.7 feet to 2.8 feet. Although the stable-low flow channels will not exist in the diversion channel, a low-flow channel was still designed for R7S2 and R7S1 by lowering the low-flow channel height that was preliminarily designed by the Corps (USACE-SFD, 2002) to approximately 1.7 feet, which is consistent with the upstream Sub-reach R7S3. The low-flow channel was also designed in Reach 4.

5.3 Comparison of the Current Hydraulic Design with 2002 Preliminary Design

As discussed above, the preliminary design (USACE-SFD, 2002) for the Llagas Creek system was modified in this updated design only for the eight sub-reaches (R7S3, R7S2, R7S1, R6, R5, R4, R14S4 and R14S2) among the 16 sub-reaches. Within these eight sub-reaches, only the cross-sections with low-flow channel planned in the preliminary design were modified with the low-flow channel dimensions, as listed in Table 5-2.

Figures 5-62 to **5-69** show the typical comparisons of the cross-sections between the modified and preliminary designs, one representative cross-section for each sub-reach. The comparisons for individual the cross-sections can be found in the HEC-RAS model for the revised project conditions. The primary difference in the channel geometry between the modified and preliminary designs mainly is in the low-flow channel section of the composite channel, while the high-flow section and the base-flow channel remain unchanged or similar.

Table 5-2. Dimension of the Low-Flow Channel

Reach	Sub-reach	Length (ft)	Longitudinal Channel gradient	Low-flow channel			Bench width (ft)
				Width (ft)	Height (ft)	Side slope	
8	R8S4	1,070	0.26%	-	-	-	-
	R8S3	2,330	0.22%	-	-	-	-
	R8S2	2,400	0.26%	-	-	-	-
	R8S1	460	0.21%	-	-	-	-
7	R7S4	1,140	0.26%	-	-	-	-
	R7S3	3,900	0.24%	25	1.7	3:1	Variable
	R7S2	2,200	0.15%	NC	1.7	NC	Variable
	R7S1	6,400	0.14%	NC	1.7	NC	Variable
6	R6S2	2,900	0.36%	-	-	-	-
	R6S1	16,400	0.31%	30	2.5	3:1	Variable
5	R5	2,100	0.31%	30	2.5	3:1	Variable
4	R4	12,370	0.20%	110	2.4	3:1	10
14	R14S4	5,800	0.29%	45	1.9	3:1	0
	R14S3	7,200	0.37%	-	-	-	-
	R14S2	4,400	0.36%	54	2.0	3:1	0
	R14S1	500	0.20%	-	-	-	-

Note: (1) NC indicates no change compared to the Corps' (2002) design.
(2) The low-flow channel of R7S2 and R7S1 are not stable.

As shown in Figures 5-62 to 5-66, a bench or benches were designed in the composite channel cross-sections for the Sub-reaches R7S3, R7S2, R7S1, R6S1 and R5 in both the preliminary design and this modified design. However, the newly designed low-flow channel has a much shallower depth and thus has a lower bench elevation for each individual sub-reach. While the benches designed in this study are approximately 1.7 feet to 2.8 feet above the base of the low-flow channel, these benches were designed at the elevations 4 feet to 7 feet above the base in the preliminary design. Lowering the bench elevation will not only increase the channel's conveyance capability, but also improve the riparian habitats if vegetation is planted on the benches. A comparison of the hydraulic designs also indicates that the newly designed low-flow channels for these sub-reaches have a base width and bank slope that are very similar to the preliminary design.

The newly designed channel for Reach 4 is much wider than the preliminary design, as shown in Figure 5-67. While the base width of this reach does not show any significant difference from the upstream branch channels (i.e., Reaches 5, 6 and 14) in the preliminary design, it is approximately equivalent to the summed width of the two upstream branches (Reaches 14 and 5) in this design. Since Reach 4 conveys water from both upstream branches, and the bottom slope of Reach 4 is much flatter than the two upstream branches, a newly designed wider channel will improve the conveyance capacity of Reach 4. Depending on the local situation, one bench or two benches on both sides of the bank were also designed in this study for some cross-sections in Reach 4. The width of the bench is 10 feet.

The preliminary hydraulic design was not modified for Sub-reaches R14S3 and R14S1. However, the preliminarily designed cross-sections for Sub-reaches R14S4 and R14S2 were modified in this design, based on the newly sized stable low-flow channel, as shown in Figures 5-68 and 5-69. Most cross-sections are required to be widened and some need to be filled in order to achieve the base width of the computed stable low-flow channel. However, the modification of either widening or narrowing is limited to 10 to 20 feet or less. No bench was designed for these sub-reaches in Reach 14.

5.4 Sediment Budget Analysis

In order to assess the potential aggradation or deposition within the study reaches, the bed material yields were calculated using the flow duration sediment transport curve method in SAM. Sediment yields were calculated in this study for (1) the average annual conditions, (2) the effective discharge, (3) the ten-percent chance exceedance (i.e.,10-year return occurrence) flood, and (4) the one-percent chance exceedance (i.e.,100-year return occurrence) flood. The average annual conditions, which are not a single flow event, are the average cumulative daily discharges that were estimated in a entire year. The conditions of the effective discharge, 10-year return occurrence and 100-year return occurrence are considered as individual flow events.

5.4.1 Updated Sediment Transport Rating Curves

The sediment transport rating curves used in this sediment budget analysis were re-calculated for the modified channel geometry with the updated low-flow channel dimensions determined in the stable channel design, as delineated in Section 5.2. Since the channel modifications were only applied to eight out of the 16 sub-reaches, the sediment transport rating curves of the effective discharge were recalculated only for these eight sub-reaches. The eight sub-reaches include R7S3, R7S2, R7S1, R6S1, R5, R4, R14S4 and R14S2. In order to accommodate larger flow discharges associated with the one-percent exceedance flood, the sediment transport rating curves were updated for all the sixteen sub-reaches using larger ranges of flow discharges. The same four sediment transport functions in SAM, which were previously applied to obtain the sediment rating curves, were also used to determine the sediment transport rating curves. These four functions are the Yang function, the Einstein total load function, the Ackers-White function, and the Engelund-Hanson function.

Figures 5-70 to **5-85** show the updated sediment transport rating curves for the 16 sub-reaches, respectively. Compared to those used in the effective discharge analysis, the updated sediment rating curves only show a very limited difference for seven out of the eight sub-reaches. This is because the new channel design mainly differs from the preliminary design only by lowering the bench elevation, while the high-flow channel, the base width and bank slope of the low-flow channel remain unchanged or are slightly modified in the seven sub-reaches. The only exception is for Reach 4, which was significantly widened in this design in order to convey the water from the upstream channels (Reaches 5 and 14) and maintain a similar flood protection standard for the upstream channels. As a result, the sediment transport capacity of Reach 4 is reduced as compared to the preliminary design.

5.4.2 Sediment Transport Yield

The sediment yields for the average annual conditions and the effective discharge were computed using the flow duration curves that were derived in the effective discharge analysis, as described in Section 5.1. The annual sediment yields computed for the average annual conditions are listed in **Table 5-3** for the sixteen sub-reaches. The sediment yields computed for the effective discharge are listed in **Table 5-4**. The sediment yields listed in these tables include those computed using the four sediment transport functions in SAM, the geometric mean values of the sediment yields based on the four functions, and the equivalent mean bulk volumes. In the tables, A-W stands for the Acker-White sediment function, and E-H for the Engelund-Hansen Formula.

Table 5-3. Computed Sediment Yield for Average Annual Conditions

Sub-reach	Water Yield (ac-ft)	Sediment Yield					
		Yang (tons/yr)	Einstein (tons/yr)	A-W (tons/yr)	E-H (tons/yr)	Mean (tons/yr)	Mean (cu yd/yr)
R8S4	1,150	329	11	578	409	171	136
R8S3	1,722	1,601	684	2,468	925	1,257	1,002
R8S2	2,011	992	94	2,652	1,154	731	582
R8S1	2,201	732	61	1,927	927	531	423
R7S4	2,201	955	61	2,301	1,165	629	501
R7S3	2,906	1,439	112	3,873	1,755	1,023	815
R7S2	3,479	430	6	916	711	202	161
R7S1	4,273	611	11	1,414	995	312	248
R6S2	6,705	5,912	424	33,199	9,550	5,310	4,229
R6S1	6,705	6,053	641	31,031	8,941	5,728	4,562
R5	6,705	6,607	903	32,380	9,015	6,460	5,145
R4	15,947	6,377	571	20,648	8,761	5,066	4,035
R14S4	5,732	3,243	264	11,181	4,293	2,532	2,017
R14S3	7,215	6,791	750	28,737	8,729	5,979	4,762
R14S2	8,496	9,014	1,106	41,662	11,673	8,344	6,646
R14S1	8,895	2,954	211	10,171	4,331	2,289	1,823

Table 5-4. Computed Sediment Yield for Effective Discharge

Sub-reach	Water Yield (ac-ft)	Sediment Yield					
		Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)	Mean (cu yd)
R8S4	55	33	1	60	37	16	13
R8S3	89	169	86	271	93	138	110
R8S2	101	102	11	281	114	77	62
R8S1	109	77	6	203	91	54	43
R7S4	109	99	7	247	114	66	53
R7S3	140	146	13	423	170	108	86
R7S2	172	44	0	98	67	18	15
R7S1	211	60	1	147	92	30	24
R6S2	346	552	48	3,428	893	534	425
R6S1	346	530	72	3,239	799	561	447
R5	346	580	100	3,313	809	628	500
R4	857	559	53	1,877	745	451	359
R14S4	284	320	27	1,110	402	249	198
R14S3	372	669	82	2,901	837	604	481
R14S2	447	888	120	4,187	1,126	842	671
R14S1	472	303	21	1,029	421	229	183

The sediment yields computed, using different sediment functions, show significant differences, which can be as large as a factor of two. Among the four functions, the Ackers-White function generally predicts the largest sediment yield while the Einstein function predicts the minimum sediment discharge. Because of the wide range of the uncertainty in the sediment yields that were computed from the four different functions, the geometric mean sediment yield was designated as the representative sediment yield for each sub-reach.

The mean sediment yield computed for the annual average conditions and the effective discharge show significant differences within the study reaches. The computed sediment yields range from 171 tons (136 cubic yards) per year at R8S4 to 8,344 tons (6,646 cubic yards) per year at R14S2 for the average annual conditions and from 18 tons (approximately 15 cubic yards) at Sub-reach R7S2 to 842 tons (671 cubic yards) at R14S2 for the effective discharge. A larger sediment yield was generally computed for a sub-reach that has a greater water yield associated with a higher discharge, and has a faster flow velocity associated with a steeper channel longitudinal profile and contracted cross-section.

The sediment yields for the ten-percent and one-percent chance exceedance floods were calculated using the synthetic hydrographs. Two synthetic hydrographs (ten-percent and one-percent occurrences) for each sub-reach were derived by respectively multiplying the discharge index of the normalized hydrographs with the ten-percent and one-percent peak flow discharges of each corresponding sub-reach, as listed in Table 3-3. The 10-year and 100-year flood hydrographs that were calculated by the Santa Clara Valley Water District for Llagas Creek at a catch point near Buena Vista Avenue (Wang, 2006), as shown in **Figure 5-86**, were used to determine the normalized hydrographs. The flow discharges of the hydrographs were divided by the peak flow discharges to obtain the normalized hydrographs, as shown in **Figure 5-87**.

The water yields, sediment yields computed using the four functions, the geometric mean values and the equivalent volumes are listed in **Table 5-5** for the synthetic ten-percent exceedance flood event, and in **Table 5-6** for the one-percent flood event, respectively. For the ten-percent flood event, while the water yields range from 366 acre-feet at the most upstream sub-reach (R8S4) to 6,244 acre-feet at the most downstream reach (R4), the sediment yields vary between 418 tons (333 cubic yards) at R8S4 and 17,551 tons (13,980 cubic yards) at R14S2. For the one-percent flood event, the water yields range from 815 acre-feet at R8S4 to 15,149 acre-feet at R4, and sediment yields from 1,444 tons (1,150 cubic yards) to 59,857 tons (47,676 cubic yards) at R14S2.

Table 5-5. Computed Sediment Yield for Ten-Percent Chance Exceedance Flood

Sub-reach	Water Yield (ac-ft)	Sediment Yield					
		Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)	Mean (cu yd)
R8S4	366	402	64	2,102	564	418	333
R8S3	492	1,438	1,004	7,827	1,238	1,934	1,540
R8S2	579	1,005	319	9,220	1,563	1,466	1,168
R8S1	659	827	266	7,400	1,344	1,216	969
R7S4	659	1,054	257	7,822	1,562	1,349	1,074
R7S3	868	1,374	398	11,222	2,036	1,880	1,497
R7S2	966	436	47	2,438	760	441	352
R7S1	1,248	627	86	4,017	1,119	702	559
R6S2	3,013	7,203	1,620	118,924	16,106	12,227	9,739
R6S1	3,013	6,187	2,358	101,962	12,072	11,576	9,220
R5	3,013	7,675	3,325	131,627	14,712	14,910	11,876
R4	6,244	8,131	3,518	95,985	14,424	14,107	11,236
R14S4	1,437	2,534	658	24,251	4,193	3,608	2,874
R14S3	2,331	6,994	2,335	98,764	12,495	11,915	9,490
R14S2	2,896	9,725	3,479	155,719	18,012	17,551	13,980
R14S1	3,167	3,795	1,248	42,577	7,041	6,138	4,889

Table 5-6. Computed Sediment Yield for One-Percent Chance Exceedance Flood

Sub-reach	Water Yield (ac-ft)	Sediment Yield					
		Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)	Mean (cu yd)
R8S4	815	1,159	259	8,243	1,757	1,444	1,150
R8S3	1,076	3,758	2,818	31,267	3,928	6,005	4,783
R8S2	1,268	2,798	1,155	36,880	4,942	4,926	3,924
R8S1	1,446	2,315	989	29,609	4,246	4,119	3,281
R7S4	1,446	2,950	953	29,963	4,796	4,483	3,571
R7S3	1,899	3,649	1,406	41,608	5,960	5,972	4,757
R7S2	2,022	1,158	186	8,441	2,139	1,404	1,119
R7S1	2,681	1,732	359	14,823	3,306	2,349	1,871
R6S2	7,085	24,980	9,561	707,129	45,694	52,706	41,980
R6S1	7,085	15,998	7,700	375,737	35,208	35,729	28,458
R5	7,085	20,378	10,459	495,520	45,167	46,734	37,223
R4	15,149	26,263	15,906	467,608	52,893	56,695	45,157
R14S4	2,709	5,762	1,840	69,393	10,250	9,319	7,422
R14S3	5,188	19,471	8,072	382,953	39,033	39,150	31,183
R14S2	6,578	27,913	12,251	637,404	58,893	59,857	47,676
R14S1	7,402	11,604	5,355	185,012	23,977	22,913	18,250

5.4.3 Sediment Budget

Assessment of sediment budget is to evaluate the balance between the sediment supply (S) and the sediment yield (Y) in a pre-defined creek reach. The sediment budget analysis of the study reaches was conducted based on the computed sediment yield for the updated channel design. However, instead of directly using the computed sediment yields, other considerations were also included in this analysis in order to reasonably assess the sediment budget of Llagas Creek.

Sub-reach Re-grouping for Sediment Budget Analysis

The sediment yield calculated using the sediment function is, in fact, the sediment transport potential or capacity that the flow can convey. The sediment transport capacity is not necessarily the actual sediment yield in the creek. This is particularly true for a relatively short creek where the upstream sediment supply significantly differs from the creek's transport capacity. When a creek's sediment supply exceeds its sediment transport capacity, aggradation or deposition would occur in this channel. Conversely, erosion would occur in the creek if the sediment supply is less than the creek's sediment transport potential. Both the deposition and erosion process would typically occur in a long longitudinal distance within a creek before the sediment yield reaches the creek's transport capacity. If the creek is not sufficiently long for the sediment yield to reach the potential through the sediment deposition or erosion process, the sediment yield at the downstream end of the creek can significantly differ from the sediment transport potential calculated by the sediment function. In such case, using the sediment transport capacity calculated by the sediment functions can give a misleading information about the real sediment yield/supply of the creek.

Based on this consideration, the sediment budget within the study reaches was evaluated in the re-grouped segments instead of strictly on a sub-reach basis. The sixteen sub-reaches were re-grouped into twelve segments. While most segments only include one sub-reach, some consist of two sub-reaches with similar creek characteristics or with a short length. By re-grouping the sixteen sub-reaches into twelve segments, each segment has a length that is sufficiently long to be more reasonable for use of the calculated sediment transport capacity as the actual sediment yield of the creek segment.

Since R8S3 is designed to be the reinforced concrete box culverts as part of the future Santa Teresa Expressway expansion between Main Avenue and Wright Avenue, this sub-reach was treated as a separate segment. The sub-reach R8S4 that is located upstream of R8S3 was also treated as a separate segment. Since R8S1 is merely 460 feet long, it joins R8S2 to form an individual segment. The four sub-reaches within Reach 7 were re-grouped into two segments. R7S4 and R7S3 were grouped into one segment, and R7S2 and R7S1 (i.e., diversion channel) were grouped into one segment. The two sub-reaches in each of the two segments have similar bottom slopes and cross-sections. Because R14S1 is only approximately 500 feet in length but has a similar deposition pattern as Reach 4, the sediment budget was evaluated jointly for these two sub-reaches. Each of the remaining sub-reaches was treated as an individual segment because the individual sub-reach is sufficiently long, has the specific channel characteristics, and/or has the opposite deposition/erosion trends.

Determination of Sediment Yield and Supply

Except for R8S3, the sediment yields of the remaining segments were represented by the geometric mean values of the calculated sediment yields (capacities), as listed in Tables 5-3 to 5-6. If two sub-reaches are included in one segment, the sediment yield capacity calculated for the downstream reach was used as the actual sediment yield of this segment. The sediment transport potential in R8S3, which consists of various concrete box culverts, will be much higher than its sediment supply, as shown in Tables 5-3 to 5-6. However, because there will be no sediment supply from the bottom of the concrete box, the sediment yield for R8S3 will be the same as its sediment supply.

The sediment supply of each segment was estimated mainly from the sediment yield of the upstream segment. However, the flow discharge and the water yield of the downstream segment are larger than the upstream segment because of the water inflow from other upstream tributaries and/or the surface runoff within this segment. As a result, the sediment supply of the downstream segment should be larger than the sediment yield from the upstream segment. In this analysis, the sediment supply of one segment was calculated as the sum of the sediment yield of the upstream segment, and additional sediment supply from other tributaries and/or surface runoff. The additional sediment supply was estimated by multiplying the extra water yield (i.e., the difference between this segment and the upstream segment) with the sediment yield that was normalized by the total water yield of this segment, assuming that the sediment concentration in the extra water supply is identical as that in this segment.

Uncertainty in Computed Sediment Budget

It is noted that the uniform flow condition is the basic assumption adopted in the hydraulic analysis using SAM. This assumption assumes the study reach has the uniform cross-sections with a constant bottom slope that equals the water surface slope. However, the designed cross-sections within each sub-reach are different, and the difference can be significant, particularly for the high-flow channels within most sub-reaches. Selecting a different cross-section within each sub-reach as the representative cross-section used in the SAM analysis will yield different sediment transport rating curves and thus different sediment budgets. A sensitive analysis indicates that the difference in the sediment budget is approximately within a range of plus or minus twenty percent when selecting different representative cross-sections. Therefore, a 15-percent threshold to assess the aggradation or erosion condition in a segment was adopted in this sediment budget analysis. If the net sediment budget (S-Y) is within 15 percent of the sediment yield, or the S/Y ratio is between 85 and 115 percent, the estimated net sediment budget (S-Y) is considered within the range of uncertainty. The sediment budget is considered as being balanced and consequently stable. Otherwise, the sediment budget is unbalanced and the channel is unstable.

Assessment of Sediment Budget

Based on the sediment yields calculated using SAM and the considerations discussed above, the sediment budget for each of the twelve segments was estimated for the average annual conditions, the effective discharge, and the ten-percent and one-percent exceedance chance flood events, as respectively listed in **Table 5-7** to **Table 5-10**. Also listed in these tables include the ratio of the sediment supply (S) to the sediment yield (Y) (i.e., S/Y), the net sediment budget defined as the sediment supply minus the sediment yield (i.e., S-Y), and the resulting channel erosion or deposition pattern. Since the sediment supply for the most upstream segments of the study reaches, i.e., R8S4 and R14S4, are not known, the net sediment yield and deposition/erosion pattern were not estimated for these two sub-reaches.

As presented in Table 5-7, the annual average sediment budget will be balanced in most of Reach 8 (i.e., R8S3, R8S2 and R8S1) and in Sub-reaches R6S1, R5, and R14S2. The sediment supply of these creek segments ranges from 87 percent to 100 percent of the corresponding sediment yield. These S/Y ratios are within the threshold range of the "balanced" sediment budget as discussed above. The channel will be stable in these sub-reaches under the average annual conditions. Since the concrete box culverts will be used in the Sub-reach R8S3, this sub-reach is non-erodible. In addition, the sediment transport potential of this sub-reach will be much larger than the supply, and thus sedimentation will not occur in this sub-reach either. In other words, Sub-reach R8S3 will be stable. The annual sediment supply was estimated to be three times or more of the sediment yield in the lower segment of Reach 7 (i.e., R7S2 and R7S1), Reach 4, and

Sub-reach R14S1. This will result in significant sedimentation within these sub-reaches. Erosion will occur in Sub-reaches R7S4, R7S3, R6S2, and R14S3 as a result of the annual sediment yield potential exceeding the annual sediment supply.

Table 5-7. Computed Sediment Budget for Average Annual Conditions

Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S/Y	S-Y (tons)	Bed Change
			Supply	Yield			
R8S4	1,070	1,150		171			
R8S3	2,330	1,722	372	372	100%	0	Stable
R8S2, R8S1	2,860	2,201	488	531	92%	-43	Stable
R7S4, R7S3	5,040	2,906	780	1,023	76%	-243	Erosion
R7S2, R7S1	8,600	4,273	1,123	312	360%	811	Deposition
R6S2	2,900	6,705	2,238	5,310	42%	-3,072	Erosion
R6S1	16,400	6,705	5,310	5,728	93%	-418	Stable
R5	2,100	6,705	5,728	6,460	89%	-732	Stable
R4, R14S1	12,870	15,947	15,041	5,066	297%	9,975	Deposition
R14S4	5,800	5,732		2,532			
R14S3	7,200	7,215	3,761	5,979	63%	-2,218	Erosion
R14S2	4,400	8,496	7,237	8,344	87%	-1,108	Stable

Table 5-8. Computed Sediment Budget for Effective Discharge

Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S/Y	S-Y (tons)	Bed Change
			Supply	Yield			
R8S4	1,070	55		16			
R8S3	2,330	89	36	36	100%	0	Stable
R8S2, R8S1	2,860	109	46	54	85%	-8	Stable
R7S4, R7S3	5,040	140	78	108	72%	-30	Erosion
R7S2, R7S1	8,600	211	118	30	394%	88	Deposition
R6S2	2,900	346	238	534	45%	-295	Erosion
R6S1	16,400	346	534	561	95%	-27	Stable
R5	2,100	346	561	628	89%	-67	Stable
R4, R14S1	12,870	857	1,504	451	333%	1052	Deposition
R14S4	5,800	284		249			
R14S3	7,200	372	392	604	65%	-212	Erosion
R14S2	4,400	447	745	842	89%	-97	Stable

As listed in Table 5-8, the sediment budget will be balanced (i.e., the sediment supply will be equivalent to the sediment yield potential) in Sub-reaches R8S2, R8S1, R6S1, and R14S2, and Reach R5 under the effective discharges that were derived in this study. The sediment supply of these creek segments ranges from 85 percent to 95 percent of the corresponding sediment yield. As a result, the channel will be stable under the effective discharge for these sub-reaches.

Table 5-9. Computed Sediment Budget for Ten-Percent Chance Exceedance Flood

Segment	Length (ft)	Water yield (ac-ft)	Sediment (tons)		S/Y	S-Y (tons)	Bed Change
			Supply	Yield			
R8S4	1,070	366		418			
R8S3	2,330	492	910	910	100%	0	Stable
R8S2, R8S1	2,860	659	1,218	1,216	100%	2	Stable
R7S4, R7S3	5,040	868	1,669	1,880	89%	-211	Stable
R7S2, R7S1	8,600	1,248	2,094	702	298%	1,392	Deposition
R6S2	2,900	3,013	7,864	12,227	64%	-4,363	Erosion
R6S1	16,400	3,013	12,227	11,576	106%	651	Stable
R5	2,100	3,013	11,576	14,910	78%	-3,334	Erosion
R4, R14S1	12,870	6,244	33,218	14,107	235%	19,111	Deposition
R14S4	5,800	1,437		3,608			
R14S3	7,200	2,331	8,178	11,915	69%	-3,737	Erosion
R14S2	4,400	2,896	15,339	17,551	87%	-2,212	Stable

Table 5-10. Computed Sediment Budget for One-Percent Chance Exceedance Flood

Segment	Length (ft)	Water yield (ac-ft)	Sediment (tons)		S/Y	S-Y (tons)	Bed Change
			Supply	Yield			
R8S4	1,070	815		1444			
R8S3	2,330	1,076	3,144	3144	100%	0	Stable
R8S2, R8S1	2,860	1,446	4,198	4,119	102%	79	Stable
R7S4, R7S3	5,040	1,899	5,544	5,972	93%	-429	Stable
R7S2, R7S1	8,600	2,681	6,658	2,349	283%	4,308	Deposition
R6S2	2,900	7,085	35,112	52,706	67%	-17,595	Erosion
R6S1	16,400	7,085	52,706	35,729	148%	16,977	Deposition
R5	2,100	7,085	35,729	46,734	76%	-11,005	Erosion
R4, R14S1	12,870	15,149	112,152	56,695	198%	55,457	Deposition
R14S4	5,800	2,709		9,319			
R14S3	7,200	5,188	28,026	39,150	72%	-11,124	Erosion
R14S2	4,400	6,578	51,799	59,857	87%	-8,058	Stable

However, under the effective discharge condition, the sediment supply will significantly exceed the sediment yield potential in Sub-reaches R7S2, R7S1 (i.e., the diversion channel), and R14S1 and Reach R4. For the diversion channel (R7S1) and R7S2, the sediment supply is almost four times of the sediment yield potential of these two sub-reaches. The sediment supply in the segment that includes both Reach 4 and Sub-reach R14S1 will be more than three times of the sediment yield. Therefore, significant sedimentation is expected to occur in these sub-reaches. Conversely, the sediment supply will be less than the sediment yield in Sub-reaches R7S4, R7S3, R6S2, and R14S3. The ratio of sediment supply to sediment yield (S/Y) ranges from 45 percent for R6S2 to 72 percent for R7S3, and the resulting erosion can be significant for R6S2 and R14S3, and may be noticeable for the upper sediment of Reach 7 (i.e., R7S4 and R7S43) under the effective discharge condition.

Table 5-9 lists the sediment budget of the study reaches for the ten-percent chance exceedance (i.e., 10-year) flood event. For the ten-percent exceedance flood event, the sediment budget will be balanced for the lower segment of Reach 8 and the upper segment of Reach 7 (i.e., from R8S3 to R7S3), R6S1 and R14S2. The sediment supply will exceed the sediment yield, resulting in channel sedimentation, in the lower segment of Reach 7 including the diversion channel, Reach 4 and Sub-reach R14S1. The sediment yield was computed to be less than the yield, resulting in channel erosion, in R6S2, R5 and R14S3.

The sediment budget for the one-percent exceedance chance (i.e., 100-year) flood is presented in Table 5-10. The sediment budget was estimated to be balanced only in R8S3 to R7S3 and R14S2 and thus the stable channel is expected in these sub-reaches. The ratio of the sediment supply to sediment yield will be approximately 148 percent for R6S1, 198 percent for R4 and R14S1, and 283 percent for R7S2 and R7S1. Sedimentation will thus occur in these sub-reaches. The channel will be eroded under the one-percent exceedance chance flood event in R6S2 and R5 because of the sediment yield potential exceeding the sediment supply.

It should be pointed out that the sediment budget was analyzed in this study on a segment basis. However, the bottom slopes and/or the channel cross-section dimensions can locally vary from one location to another even within each sub-reach (segment). The difference can be significant for some sub-reaches (segments). Even channel erosion was estimated for one sub-reach, local sedimentation may still occur in the locations where the channels have a flatter bottom slope and/or a wide cross-section. On the other hand, local erosion may occur for the locations with a steep bottom slope and/or a contracted cross-section within a sub-reach that was estimated to have an overall sedimentation pattern. Therefore, the estimated sediment budget only represents the overall sediment budget within an individual sub-reach.

5.5 Discussion of the Diversion Channel Design

The longitudinal creek profile that was preliminarily designed (USACE-SFD, 2002) is shown in **Figure 5-88**. The proposed diversion channel is located in Sub-reach R7S1. The bottom slope of each sub-reach, based on the design creek profile, is presented in **Table 5-11**.

Table 5-11. Lengths and Bottom Slopes of Reaches 4 to 8

Reach	Sub-reach	Length	Bottom slope
8	R8S4	1,070	0.26%
	R8S3	2,330	0.22%
	R8S2	2,400	0.26%
	R8S1	460	0.21%
7	R7S4	1,140	0.28%
	R7S3	3,900	0.28%
	R7S2	2,200	0.15%
	R7S1	6,400	0.14%
6	R6S2	2,900	0.37%
	R6S1	16,400	0.31%
5	R5	2,100	0.32%
4	R4	12,370	0.19%

The design channel longitudinal gradient is approximately 0.21 percent to 0.28 percent for the sub-reaches upstream of R7S2, flattens to approximately 0.14 to 0.15 percent in R7S2 and R7S1 (i.e., the diversion channel), steepens to more than 0.30 percent in the downstream Sub-reaches R6S1 and R5, and then flattens again to 0.19 percent in Reach 4 that is downstream of the confluence of East Little Llagas Creek.

The abruptly flattened bottom slope at Sub-reaches R7S1 and R7S2 will induce significant sedimentation within these two sub-reaches. The sediment budget analysis indicated that the sediment supply would significantly exceed the sediment yield potential of the newly proposed diversion channel. The sediment supply ranges from three to four times of the sediment transport potential of the diversion channel for different flow scenarios. Based on the annual sediment budget analysis (see Table 5-7), approximately 811 tons sediment, or 70 percent of the sediment coming from the upstream reach, will annually deposit in the newly constructed diversion channel (R7S1) and in the sub-reach upstream of the diversion channel (R7S2). The total length of R7S1 and R7S2 is 8,600 feet. The mean annual deposition rate is thus approximately 0.094 tons per liner foot length along the channel.

Because more sediment will deposit in the upper portion of the diversion channel than the downstream segment, the bottom slope of the diversion channel will gradually increase as the sediment deposition continues. This will gradually increase the sediment transport capacity of the diversion channel. It is possible that the sedimentation will eventually stop and the diversion channel will reach a quasi-stable status. However, it will take a long time for the diversion channel to reach the stable status. By the time, the flow capacity of the diversion channel may be significantly reduced and the level of flood protection will be lower than the original design. In order to maintain the design flow capacity of the diversion channel, a considerable amount of maintenance work will be required to remove the deposited sediment, which may induce various adverse impacts to the existing local riparian habitats.

Within Sub-reach R7S1, Middle Avenue and Watsonville Road would cross the diversion channel. Both of these bridges have already been built by SCVWD, based on the NRCS design. The proposed channel cross-sections under these two bridges have similar dimensions to other sections of the diversion channel. Therefore, local channel scouring will not likely occur next to these structures.

Because of the potential sedimentation in the diversion channel area, a stable low-flow channel will not exist in Sub-reaches R7S2 and R7S1. As discussed in the stable channel design section (i.e., Section 5.2), the design bottom slope of 0.14 percent to 0.15 percent is so flat that it is out of the slope range in the stable channel slope-width solutions calculated by SAM.

In order to alleviate the channel sedimentation and design a stable low-flow channel applicable to the diversion channel, the bottom slope of the diversion channel can be reconfigured. The objective of the bottom slope reconfiguration is to reduce the difference in the bottom slope between the diversion channel and the adjacent upstream and downstream reaches. Two preliminary options are briefly discussed in the following. A detailed supplemental study to exploit various mitigation options as well as to select a preferred option is presented in Appendix C.

5.5.1 Mitigation Option 1: Re-grading the Diversion Channel

One approach of reconfiguring the bottom slope is to re-design the channel invert elevation in Sub-reaches R6S2, R7S1 (i.e., the diversion channel) and R7S2. Figure 5-88 shows, as an example, an option of reconfiguring the bottom slope in the diversion channel area. This option will steepen the bottom slope in Sub-reaches R7S1 and R7S2, and flatten the bottom slope in Sub-reach R6S2 as well as a small portion of R6S1. The reconfigured channel will be tied into the existing upstream channel at the West Edmundson Culvert in R7S2, and tied into the downstream channel at the Llagas Ave. Bridge in R6S1. The channel reconfiguration consists of steepening of the flat segments (R7S1 and R7S2) and flattening of the steep segment (R6S2). The reconfigured channel bottom slope will be approximately 0.21 percent, compared to 0.28 percent of the upstream channel (R7S3) and 0.31 percent of the downstream channel (R6S1).

This reconfiguration option will involve (1) excavation of the existing channel (mainly R6S2 and R7S2), (2) over-excavation of the diversion channel as compared to the preliminary design plan (USACE-SFD, 2002), and (3) replacement of several bridges and culverts. The maximum excavation depth will be approximately 9 feet in R6S2. The bridges and culverts that will be impacted under this reconfiguration option as well as the excavation depth estimated at these locations are listed in **Table 5-12**. It is worthy to point out that excavation of the existing channel at R6S2 may have the adverse impacts to the endangered species that are presently identified in this sub-reach. The channel excavation may also require the overbank widening in order to maintain a stable channel bank slope.

Table 5-12. Excavation Depth at Bridges and Culverts

Sub-reach	Bridge/Culvert	Excavation Depth (ft)
R7S2	La Crosse Drive, North Bridge	2.0
	La Crosse Drive, South Bridge	2.4
R7S1	Watsonville Road	3.0
	W. Middle Ave.	5.6
R6S2	Monterey Road	3.7
	Union Pacific Railroad	3.7

If the re-grading is only limited within R7S1 and R6S2, i.e., from North Bridge of La Crosse Drive to Monterey Road, the bottom slope of these sub-reaches will be approximately 0.19 percent, and only the bridges at Watsonville Road and at W. Middle Ave. will be impacted. However, the mild bottom slope (0.15 percent) of R7S2 and the steeper bottom slope (0.35 percent) immediately downstream of Monterey Road may result in the considerable amount of sedimentation in R7S2 and erosion in R6S2.

5.5.2 Mitigation Option 2: Realigning the Diversion Channel

Another option of reconfiguring the bottom slope is to realign the diversion channel. Under this option, the confluence between the diversion channel and Llagas Creek will move downstream to the Monterey Road Bridge. By doing that, the diversion channel will have a steeper bottom slope, and will bypass Sub-reach R6S2, eliminating the adverse impacts on the existing habitats in R6S2.

The diversion channel realignment alone will create a channel bottom slope of approximately 0.19 percent. However, it is recommended that the channel realignment be combined with excavation of Sub-reaches R7S2 and R6S1 downstream of the Monterey Road Bridge. The excavation depth in the two segments will be similar to Option 1. By doing that, the bottom slope of the diversion channel and these two segments will be approximately 0.21 percent, which can smoothly be tied into the upstream and downstream reaches.

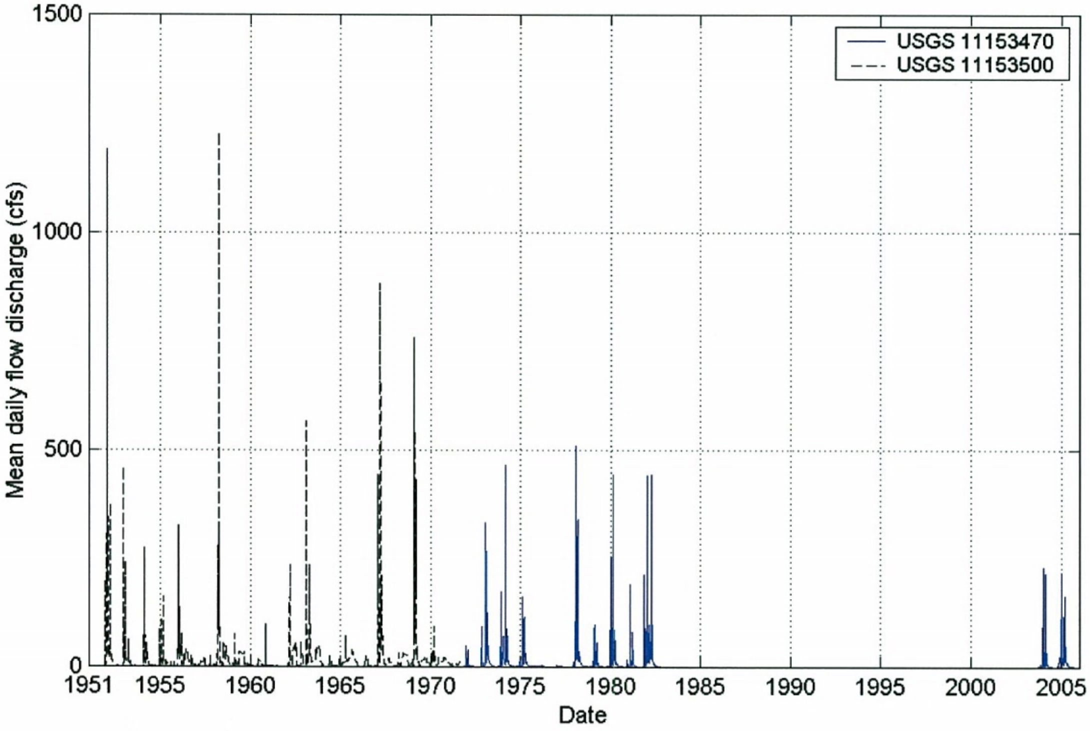


Figure 5-1. Measured Mean Daily Flow Hydrographs at Gages 11153470 & 11153500

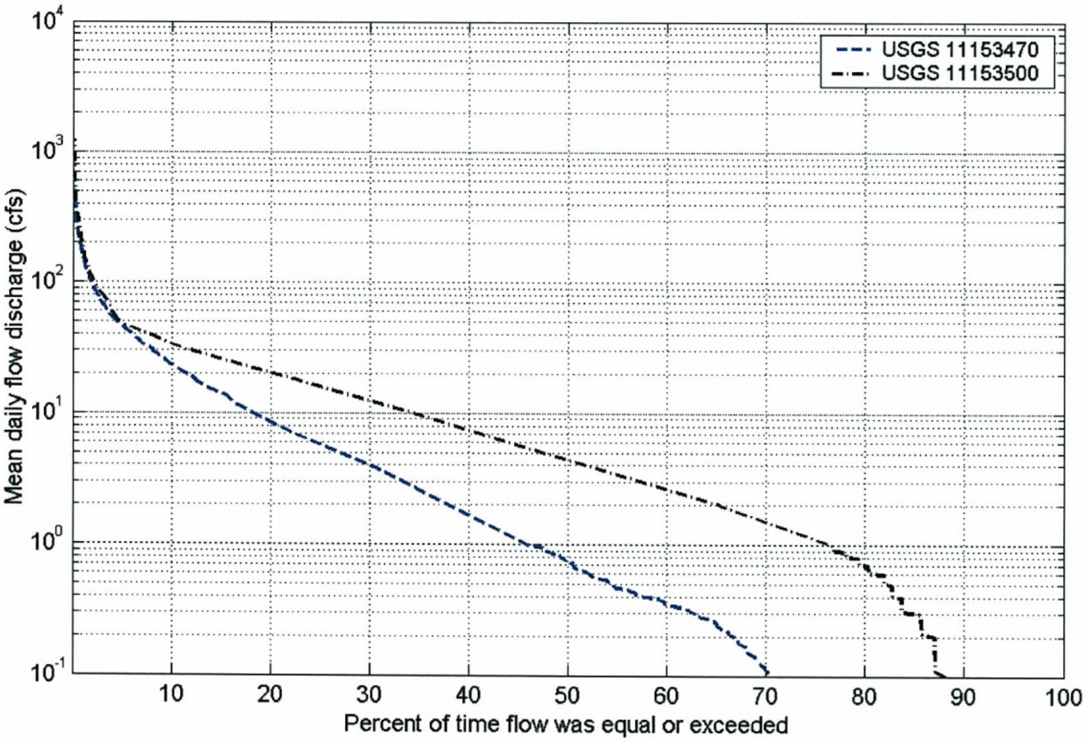


Figure 5-2. Flow Duration Curves at USGS Gages 11153470 and 11153500

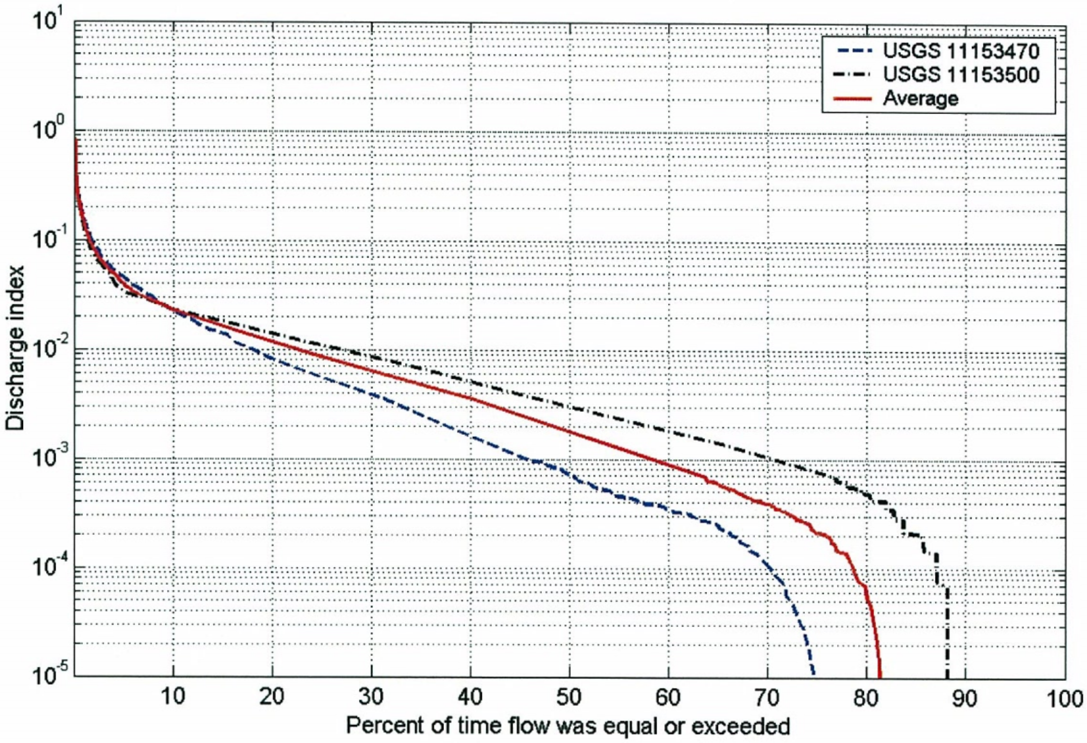


Figure 5-3. Derived Regionalized Flow Duration Curves for Gages 11153470 and 11153500, and the Site-Average Regionalized Flow Duration Curve

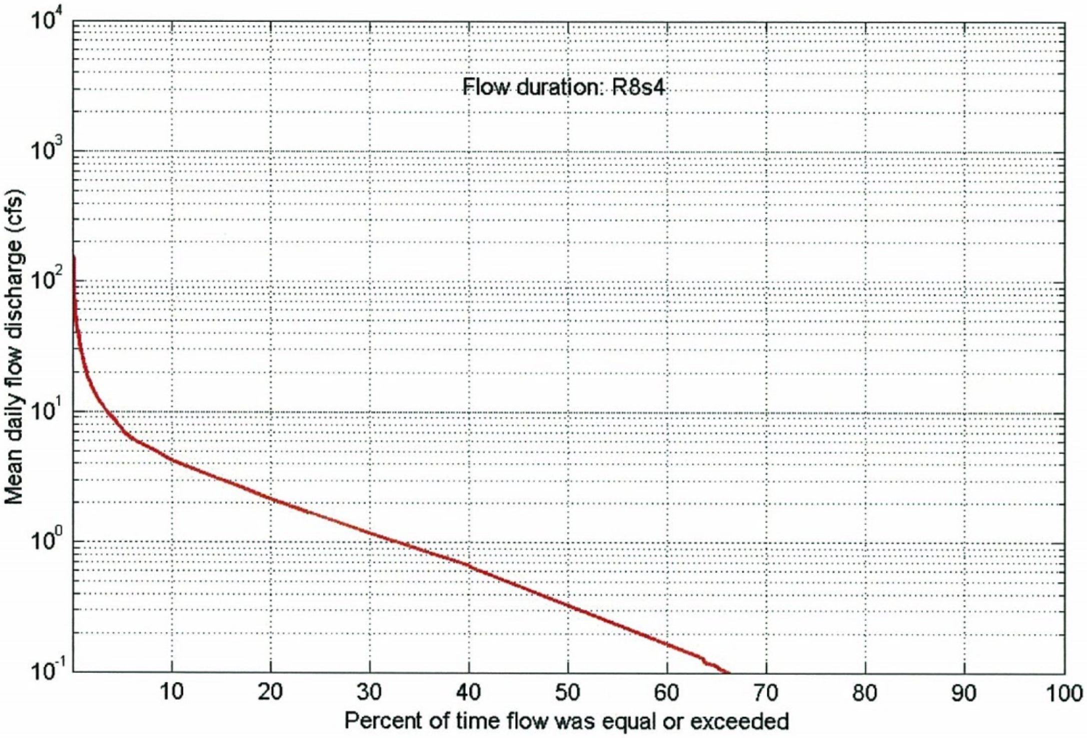


Figure 5-4. Calculated Flow Duration Curve for Sub-reach R8S4

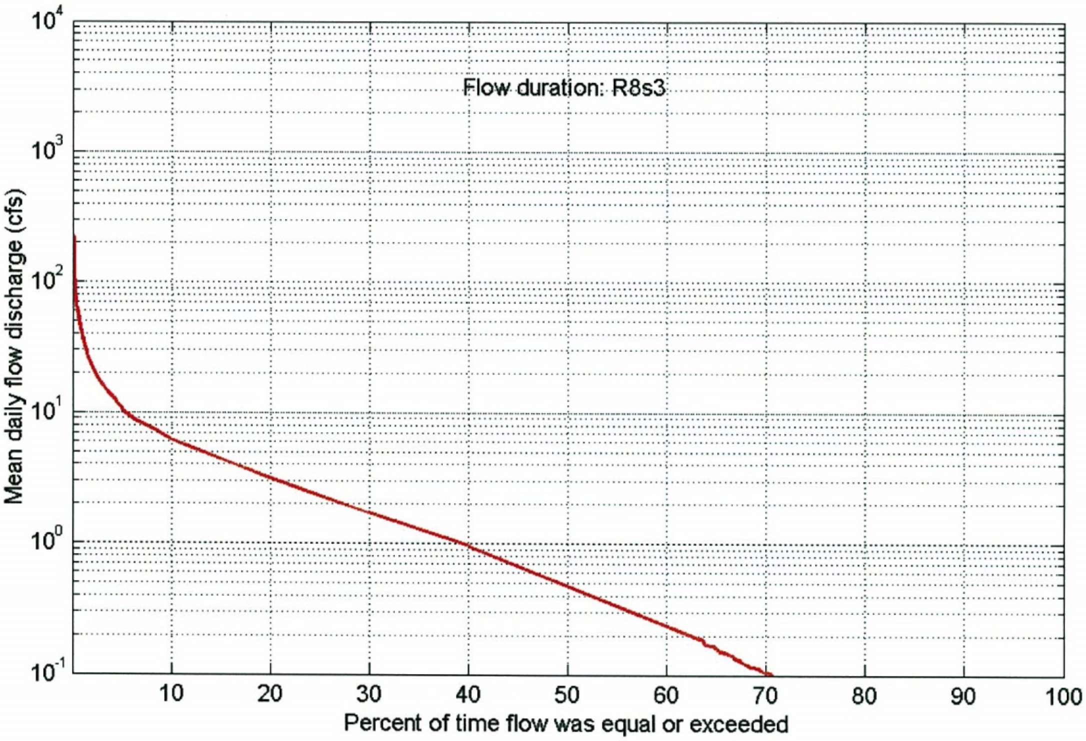


Figure 5-5. Calculated Flow Duration Curve for Sub-reach R8S3

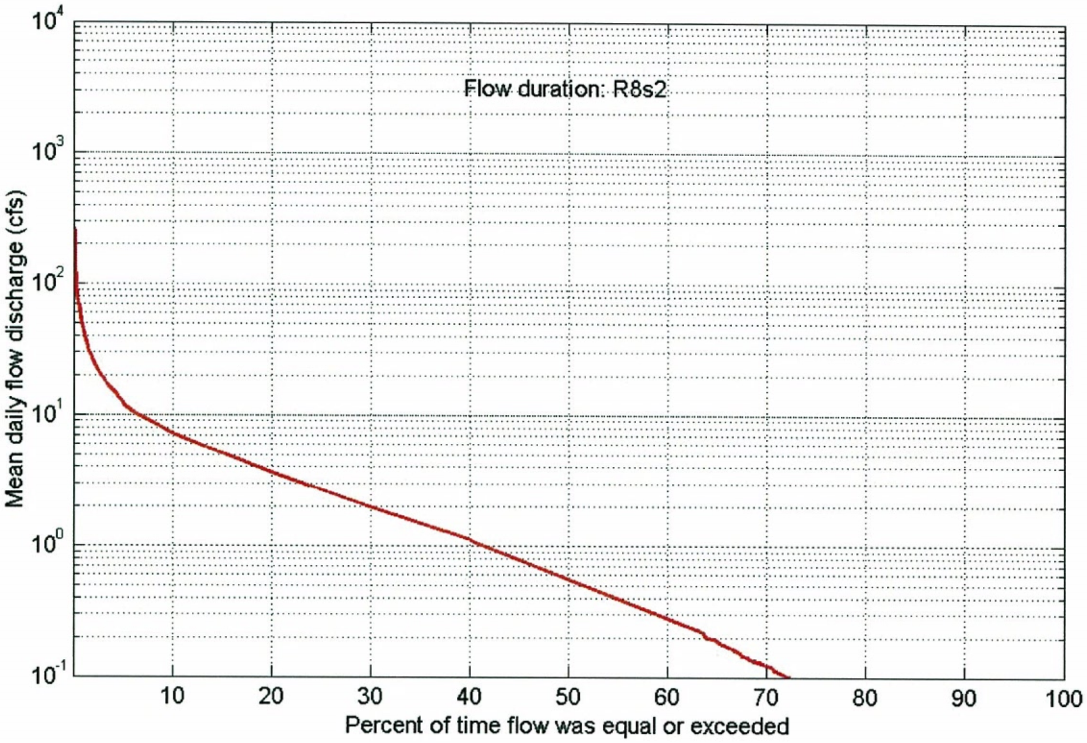


Figure 5-6. Calculated Flow Duration Curve for Sub-reach R8S2

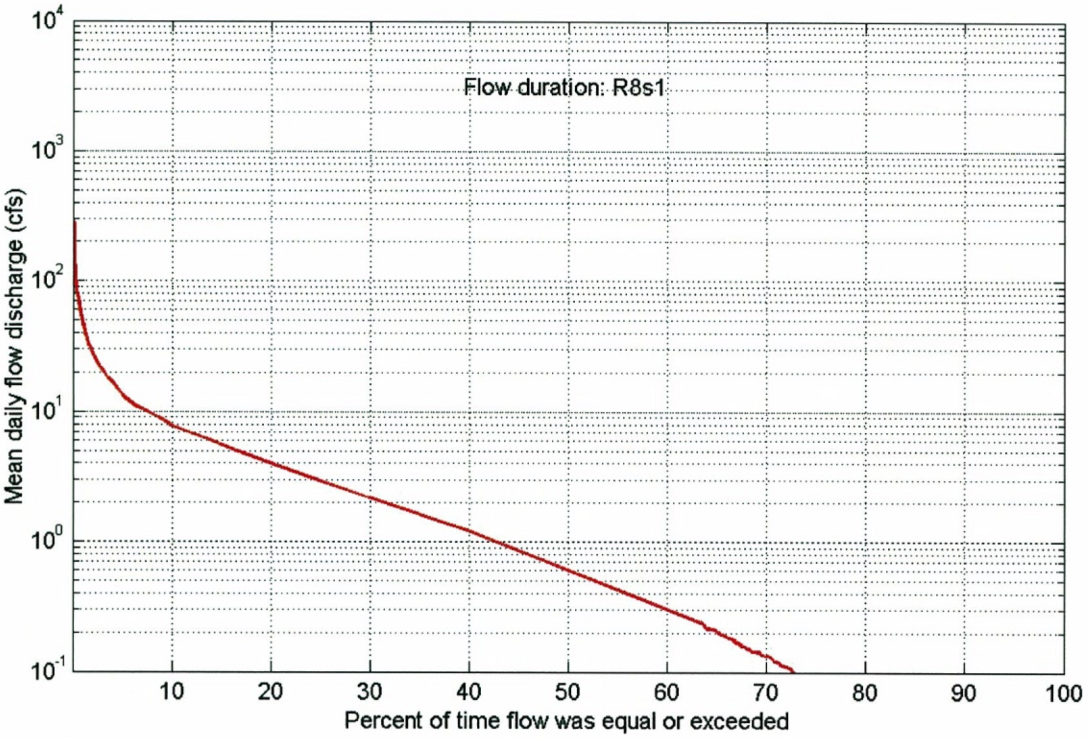


Figure 5-7. Calculated Flow Duration Curve for Sub-reach R8S1

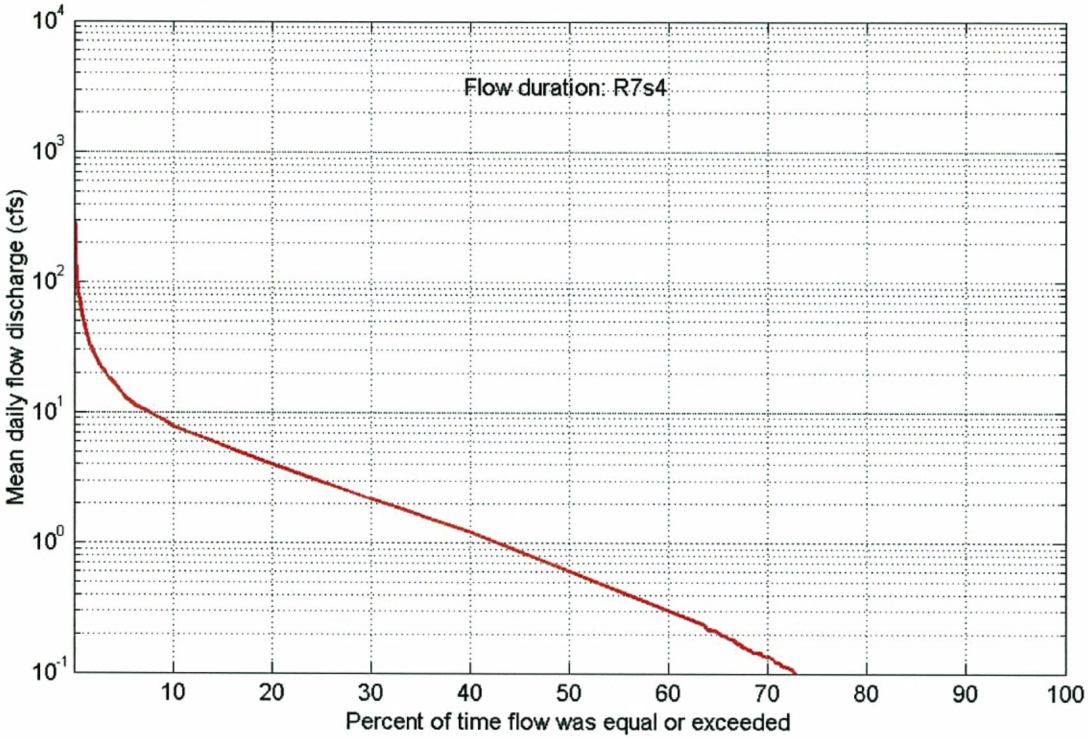


Figure 5-8. Calculated Flow Duration Curve for Sub-reach R7S4

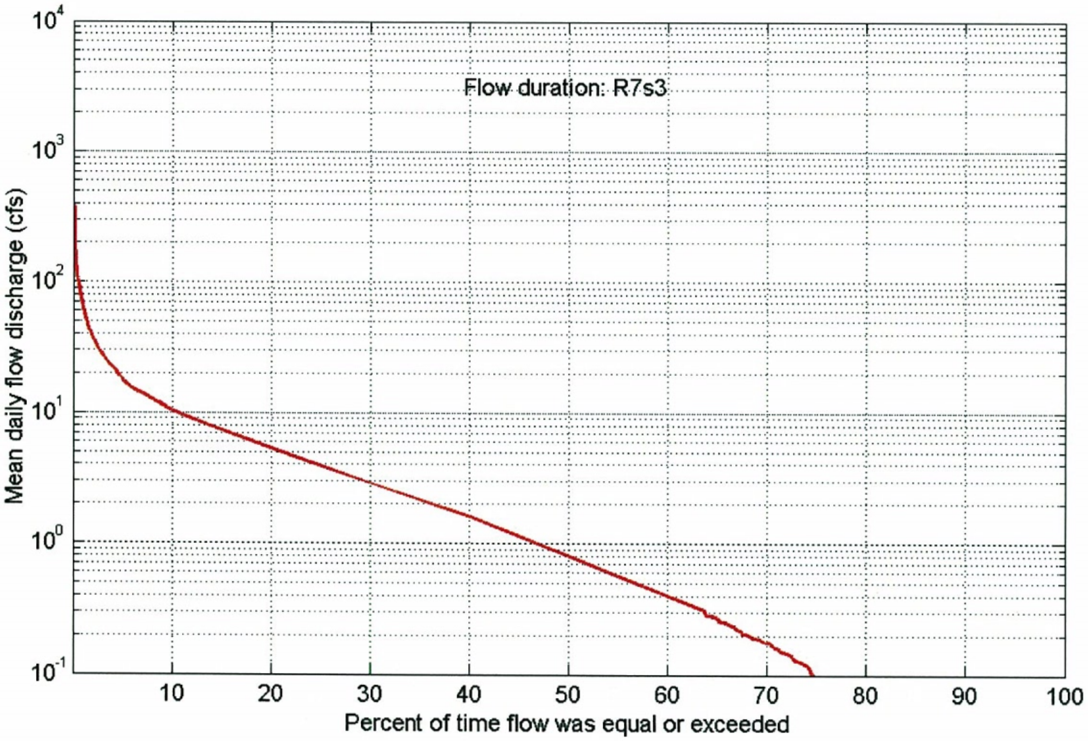


Figure 5-9. Calculated Flow Duration Curve for Sub-reach R7S3

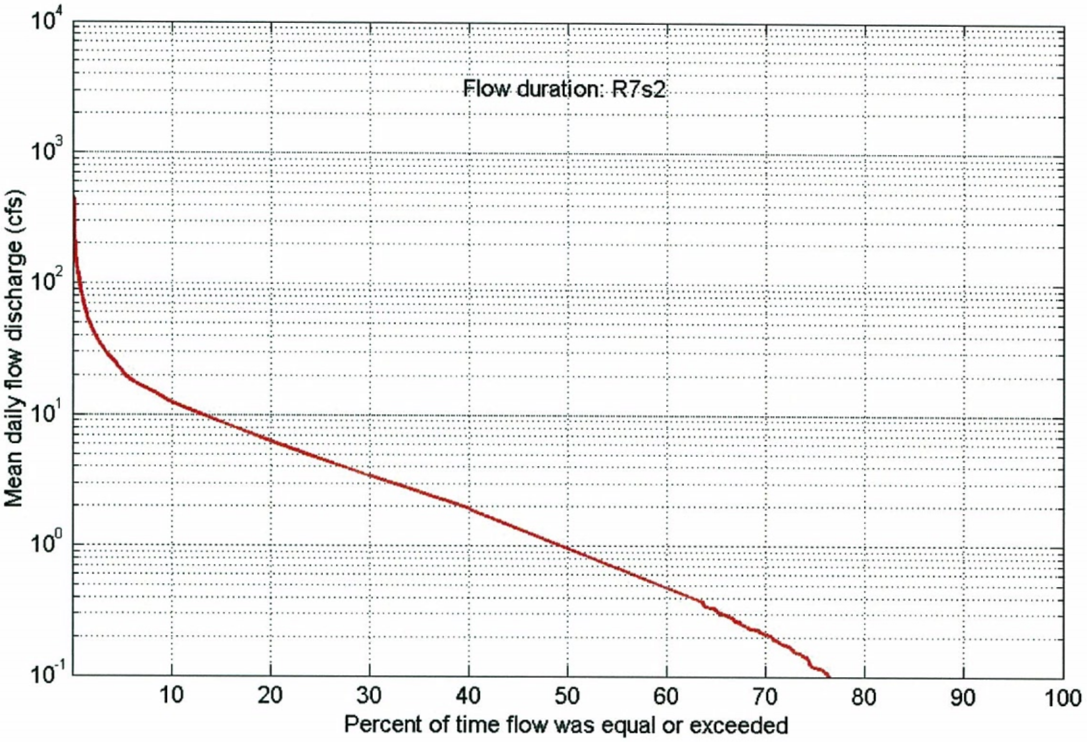


Figure 5-10. Calculated Flow Duration Curve for Sub-reach R7s2

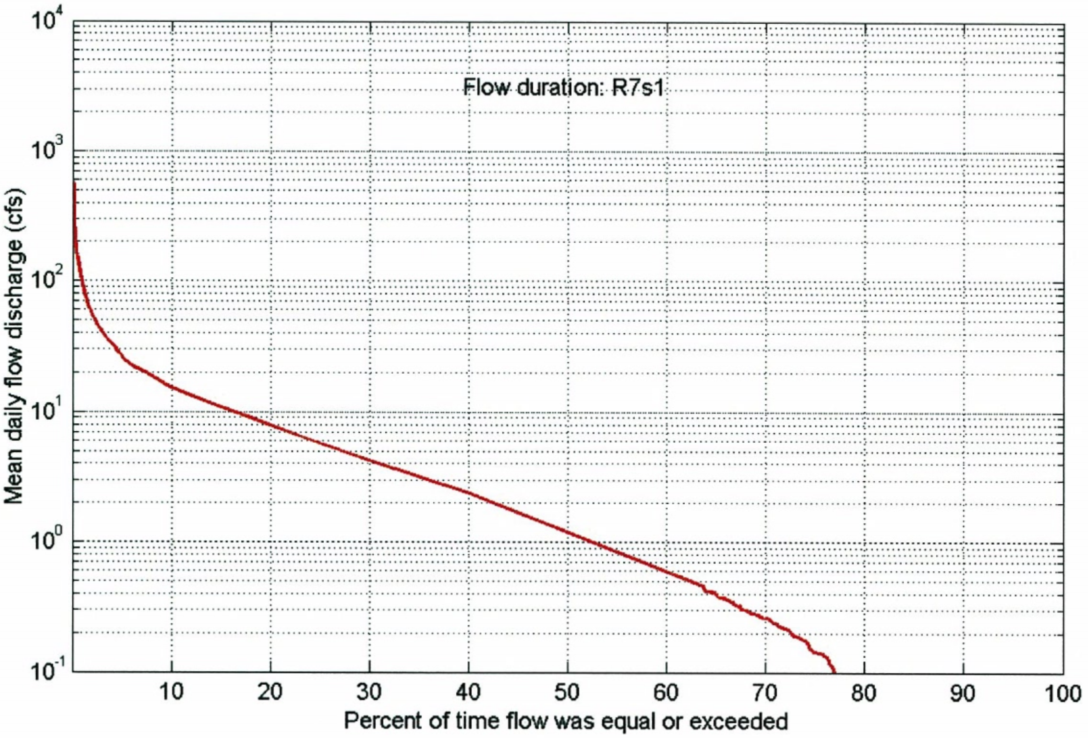


Figure 5-11. Calculated Flow Duration Curve for Sub-reach R7S1

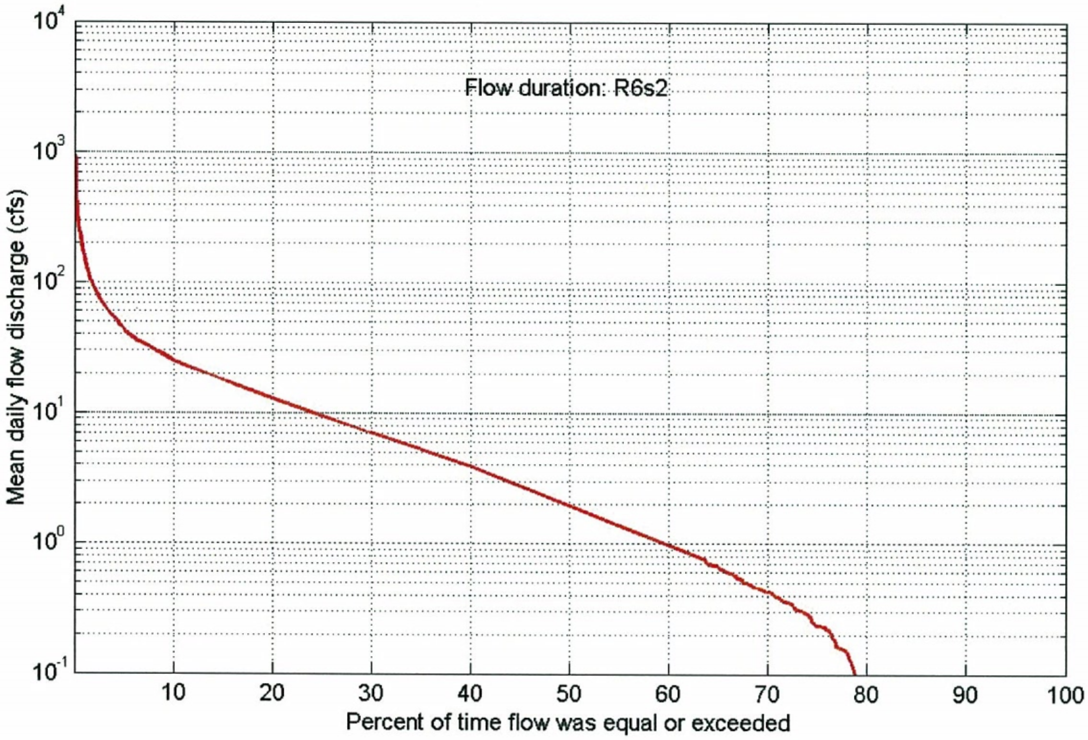


Figure 5-12. Calculated Flow Duration Curve for Sub-reach R6S2

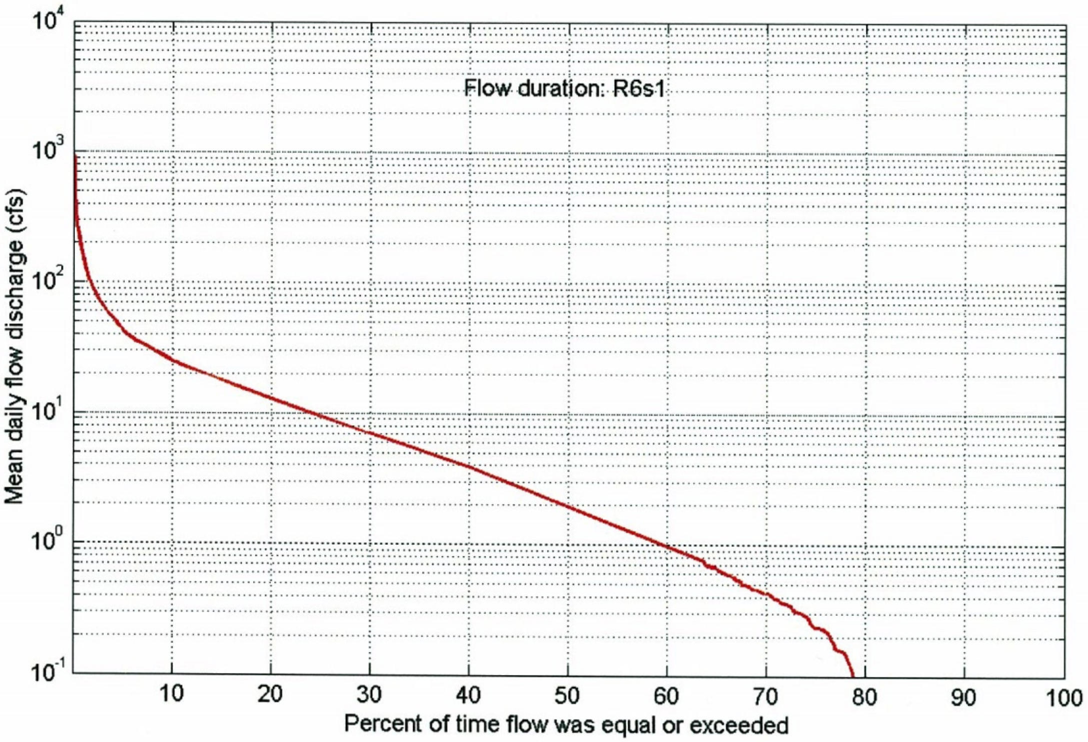


Figure 5-13. Calculated Flow Duration Curve for Sub-reach R6S2

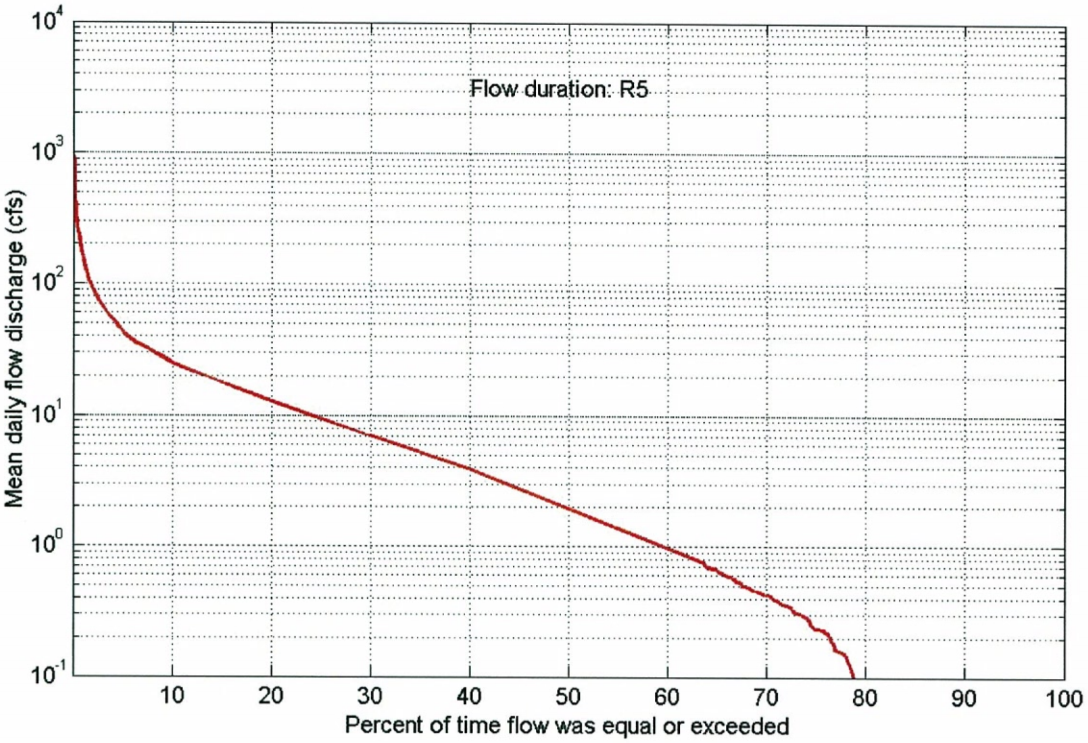


Figure 5-14. Calculated Flow Duration Curve for Reach R5

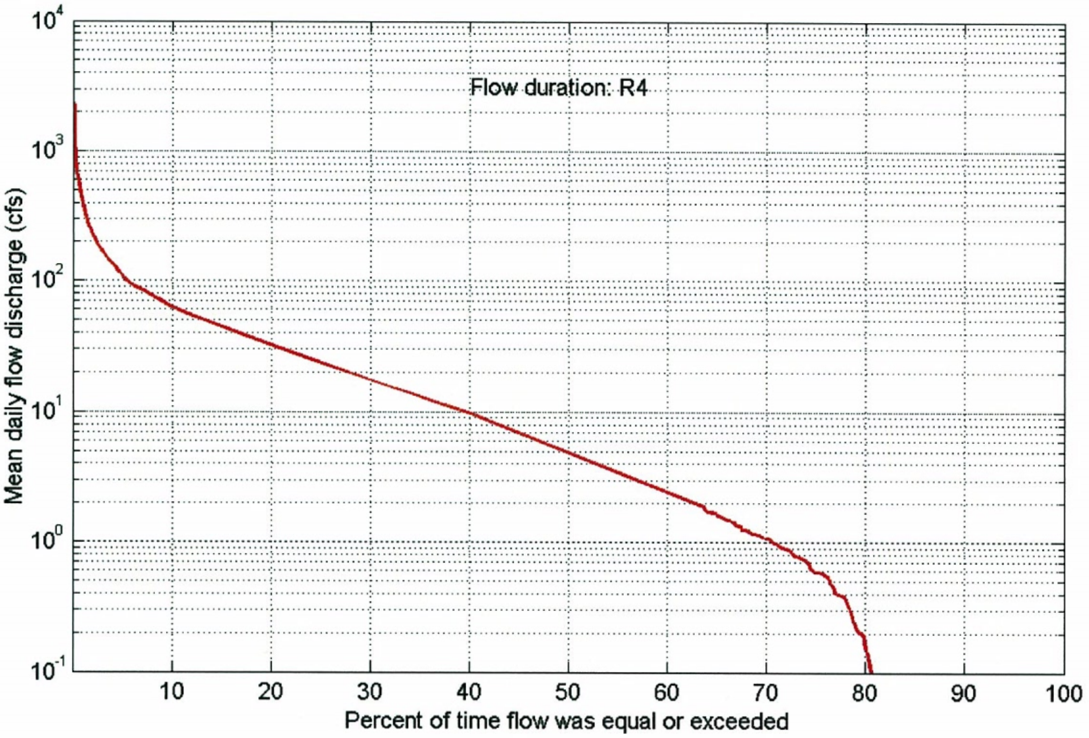


Figure 5-15. Calculated Flow Duration Curve for Reach R4

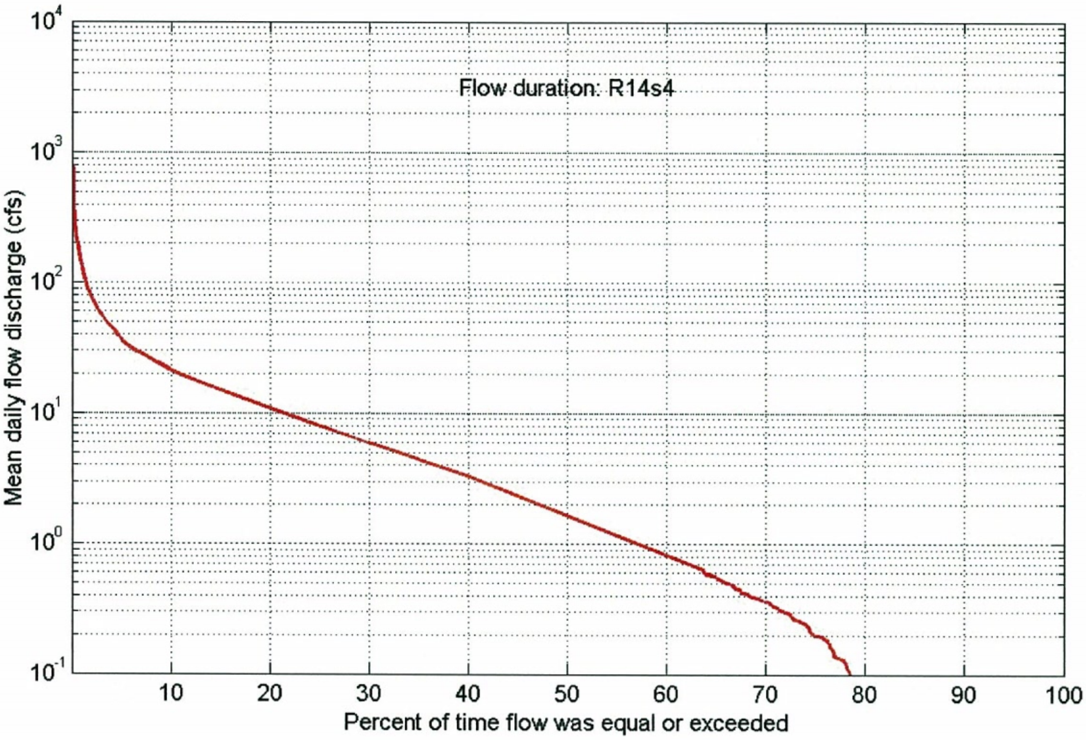


Figure 5-16. Calculated Flow Duration Curve for Sub-reach R14S4

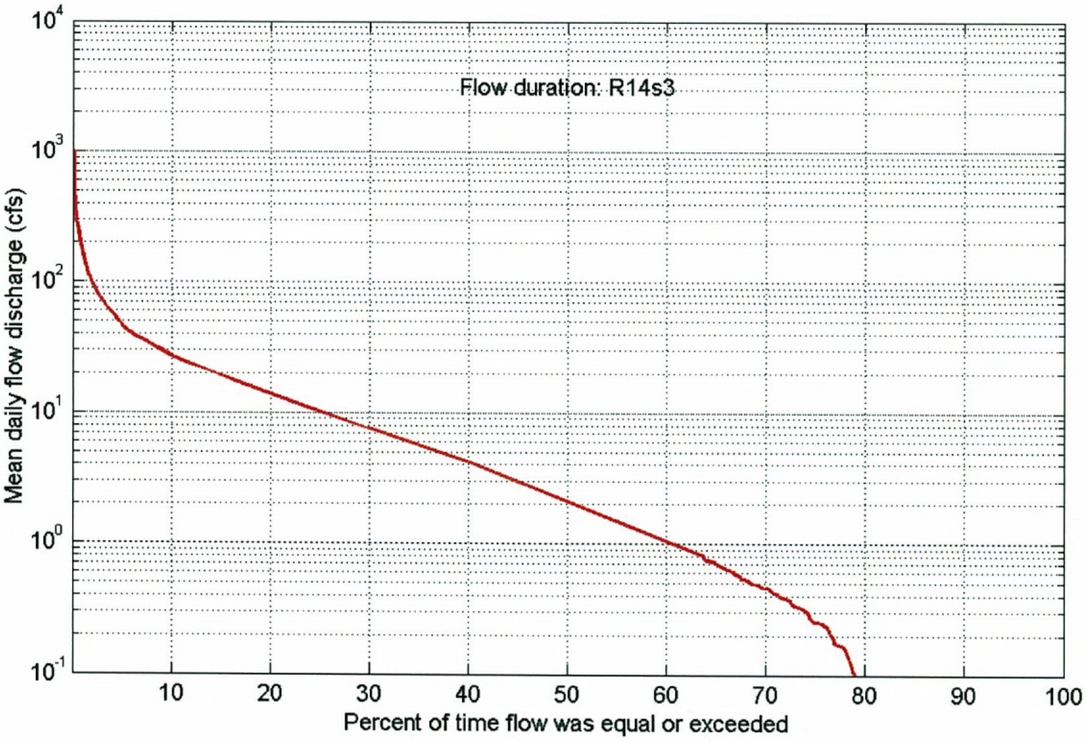


Figure 5-17. Calculated Flow Duration Curve for Sub-reach R14S3

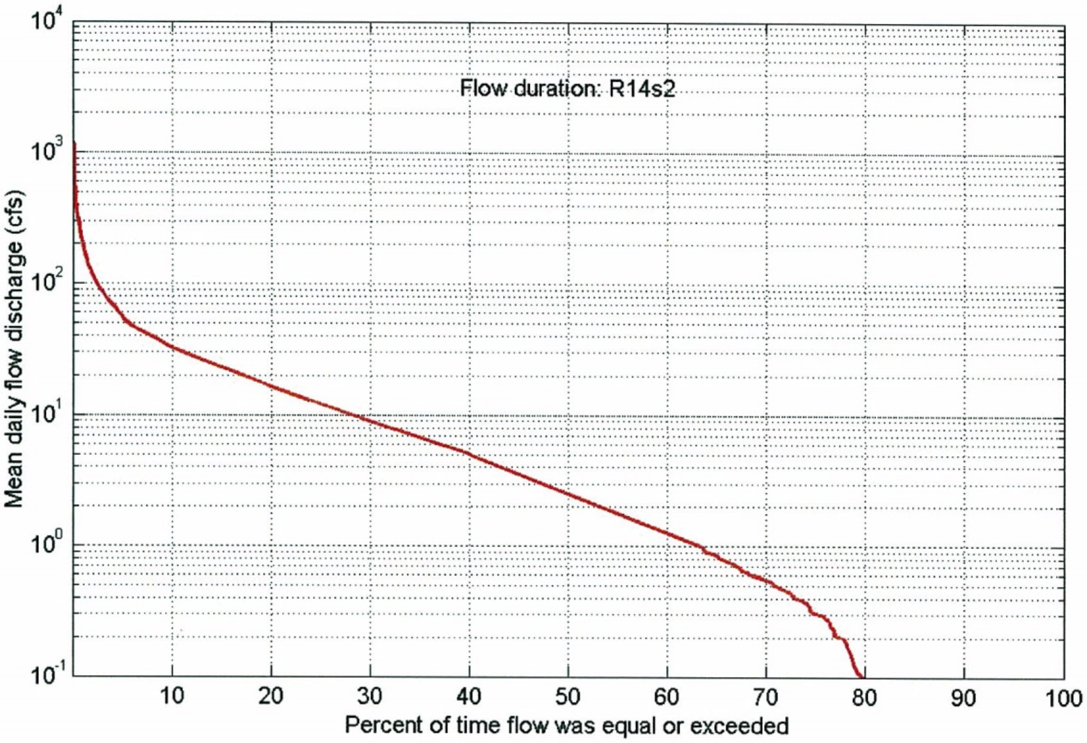


Figure 5-18. Calculated Flow Duration Curve for Sub-reach R14S2

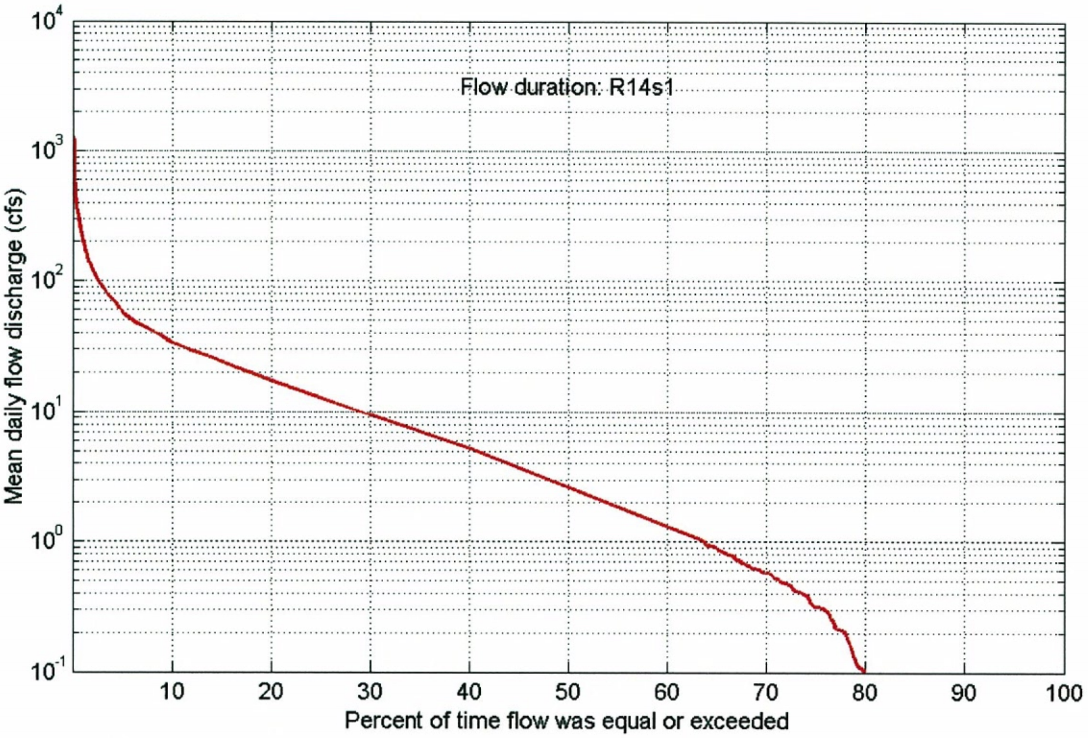


Figure 5-19. Calculated Flow Duration Curve for Sub-reach R14S1

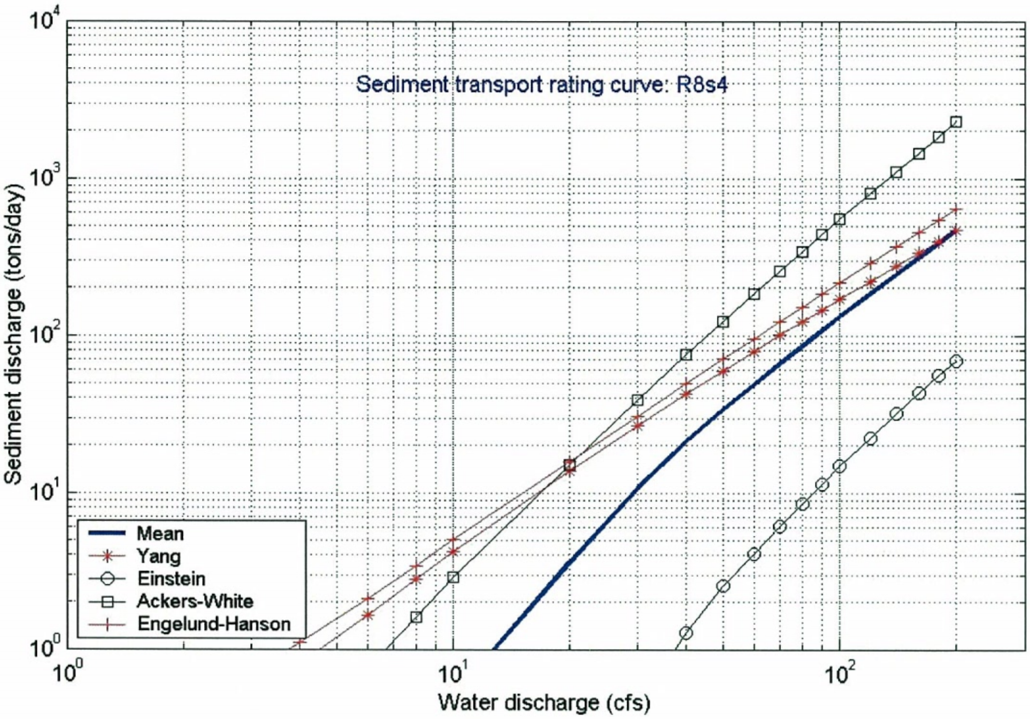


Figure 5-20. Sediment Transport Rating Curve for Sub-Reach R8S4

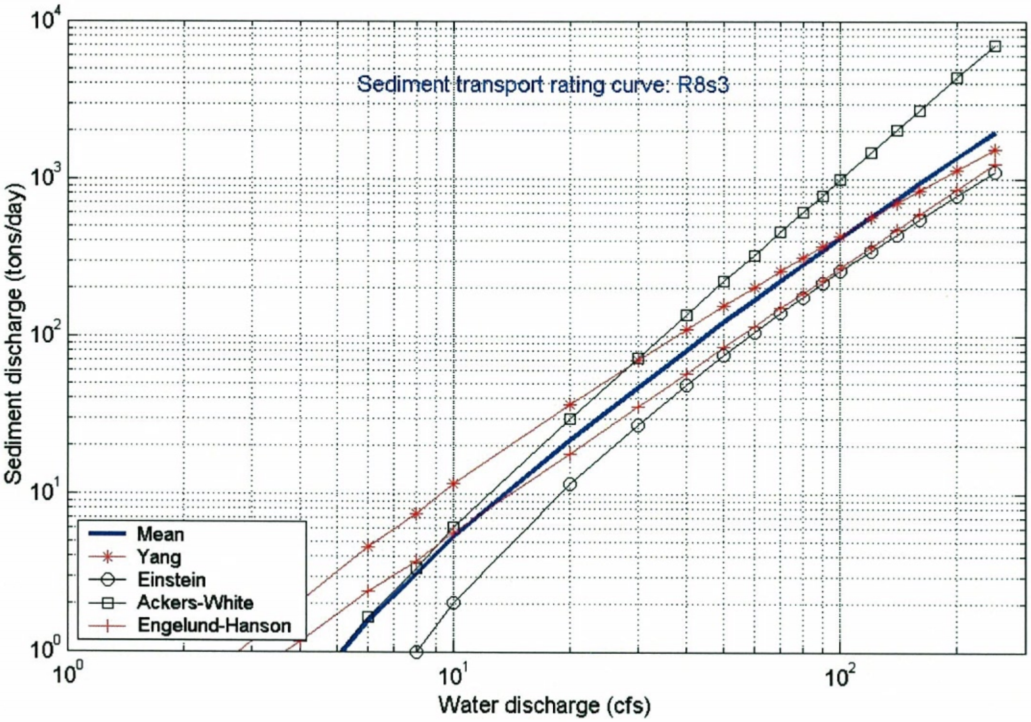


Figure 5-21. Sediment Transport Rating Curve for Sub-Reach R8S3

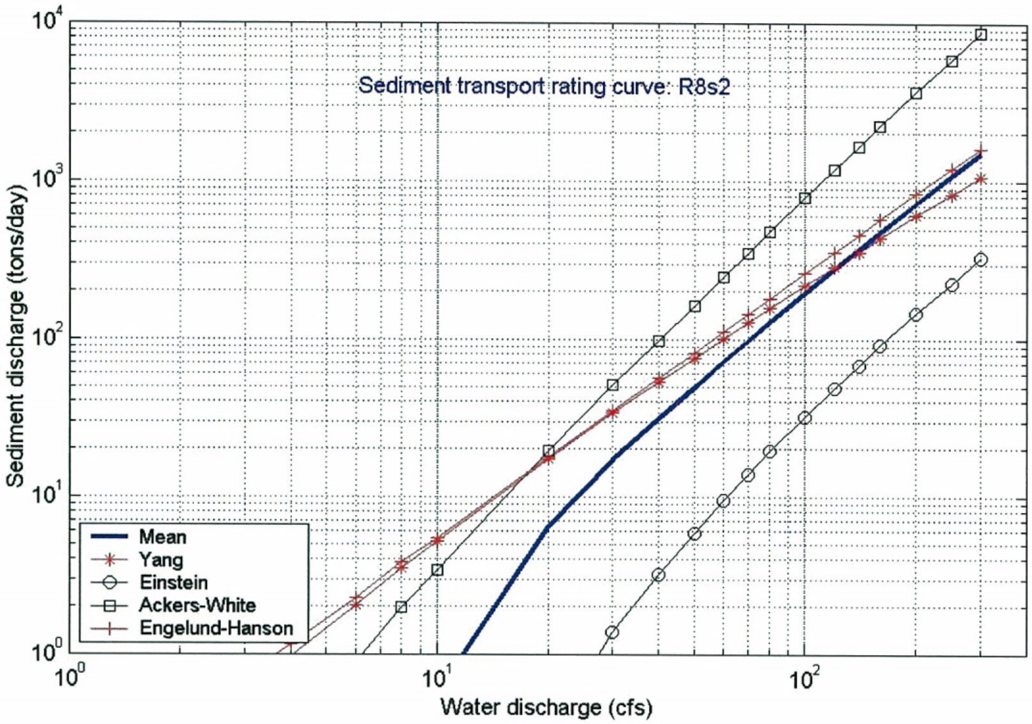


Figure 5-22. Sediment Transport Rating Curve for Sub-Reach R8S4

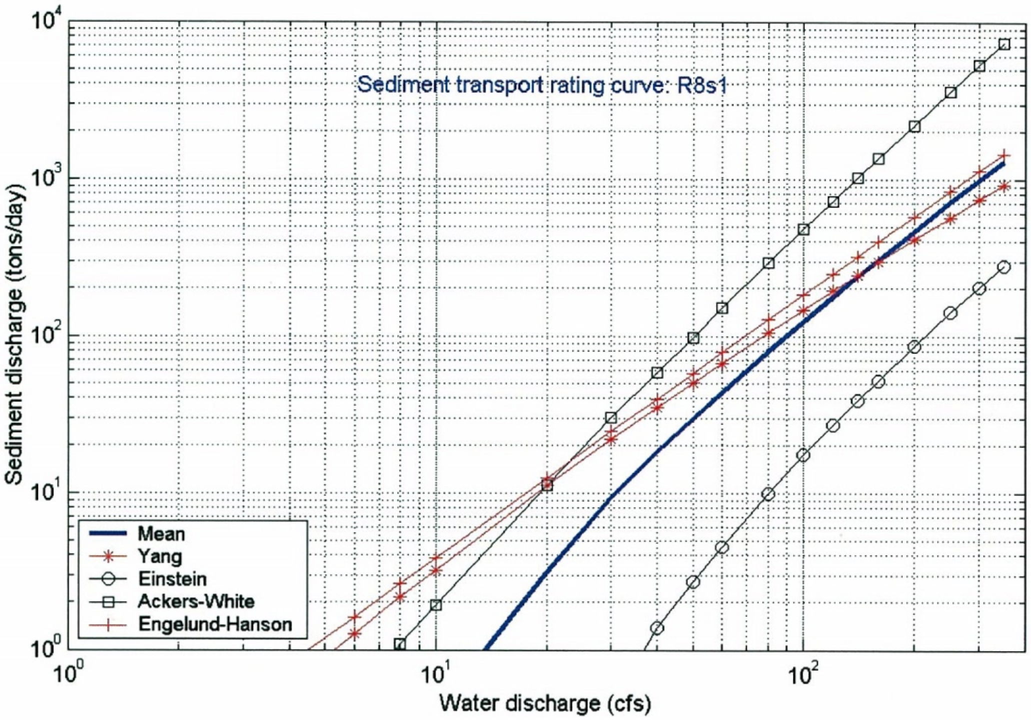


Figure 5-23. Sediment Transport Rating Curve for Sub-Reach R8S1

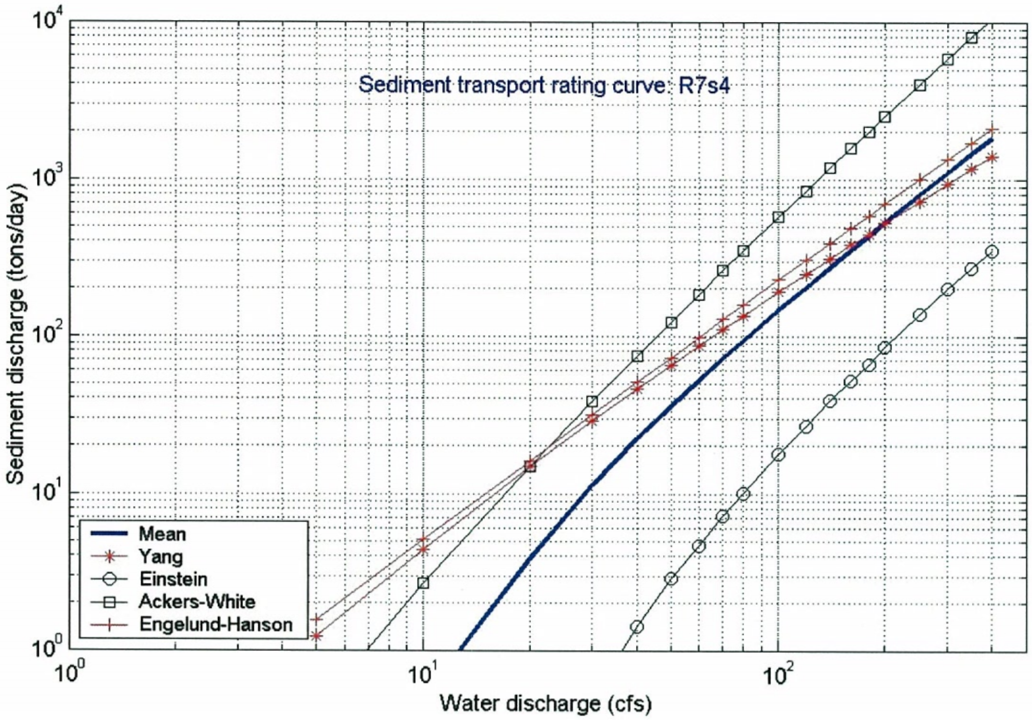


Figure 5-24. Sediment Transport Rating Curve for Sub-Reach R7S4

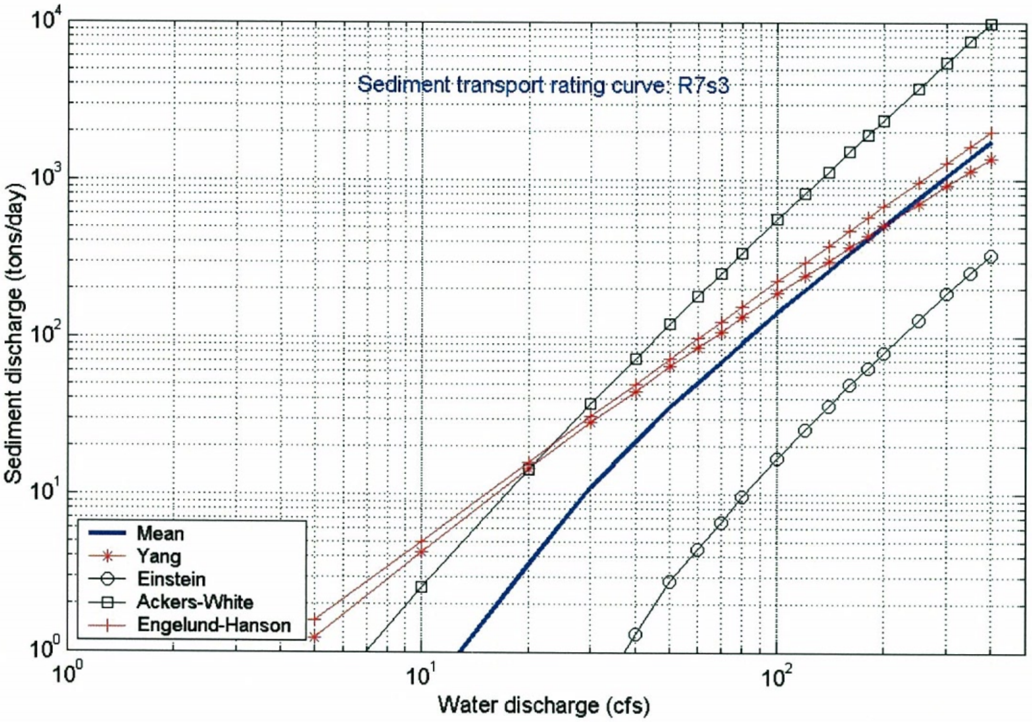


Figure 5-25. Sediment Transport Rating Curve for Sub-Reach R7S3

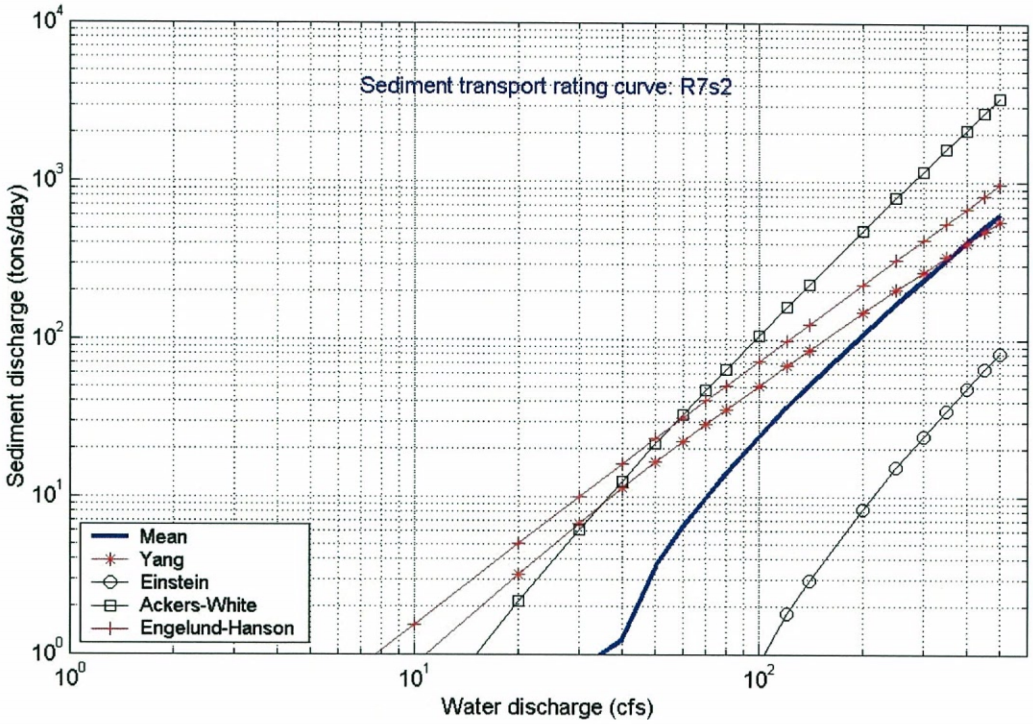


Figure 5-26. Sediment Transport Rating Curve for Sub-Reach R7S2

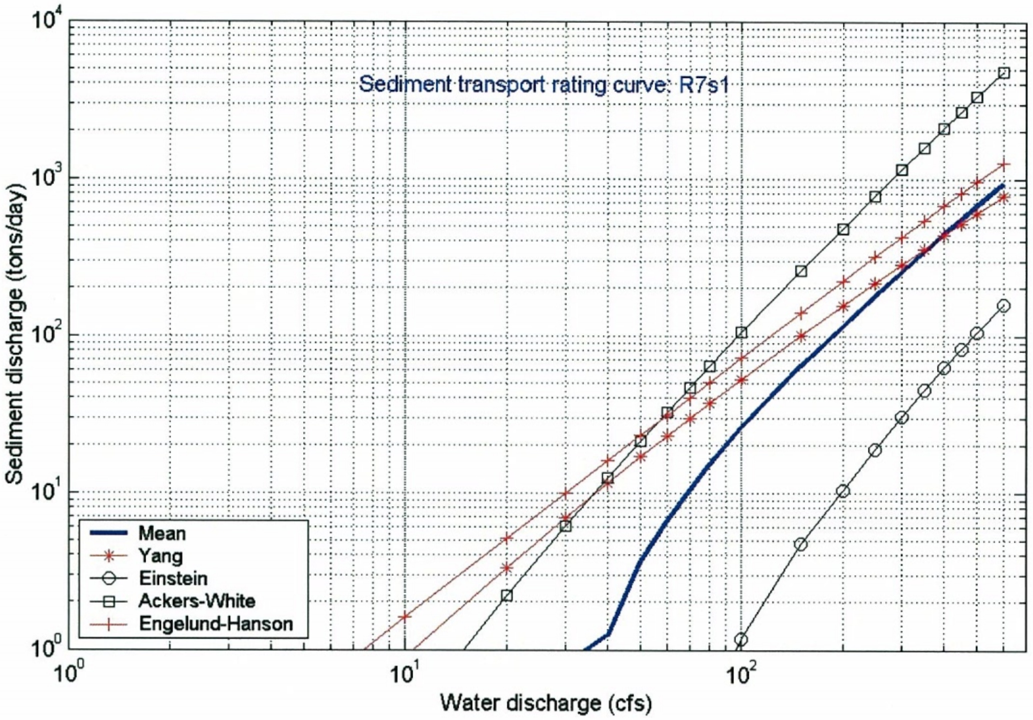


Figure 5-27. Sediment Transport Rating Curve for Sub-Reach R7S1

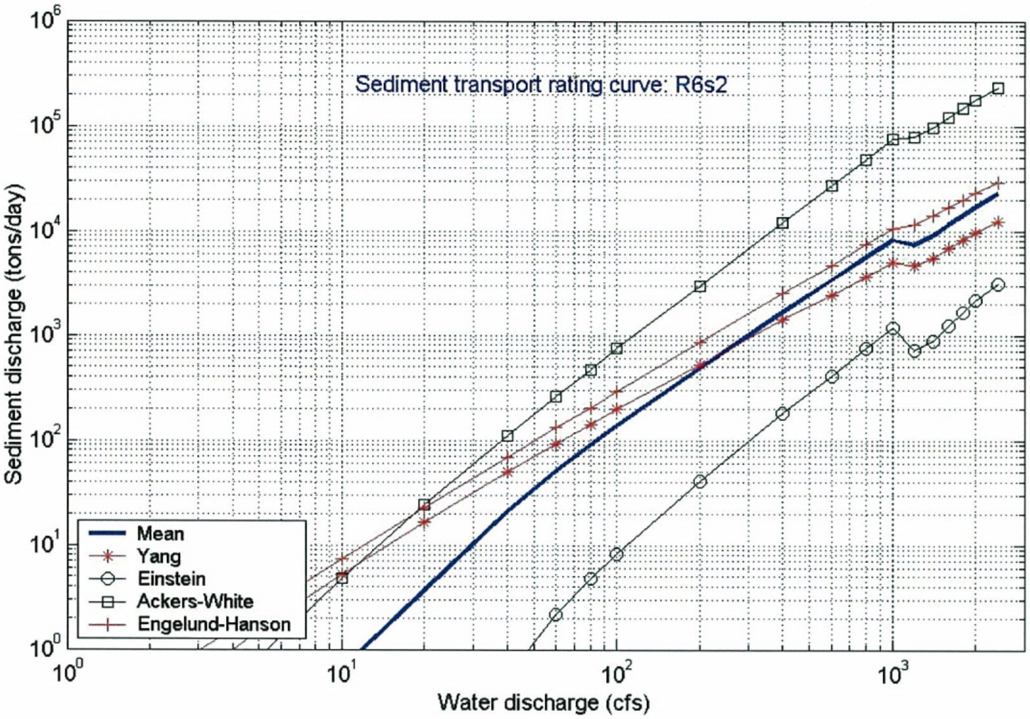


Figure 5-28. Sediment Transport Rating Curve for Sub-Reach R6S2

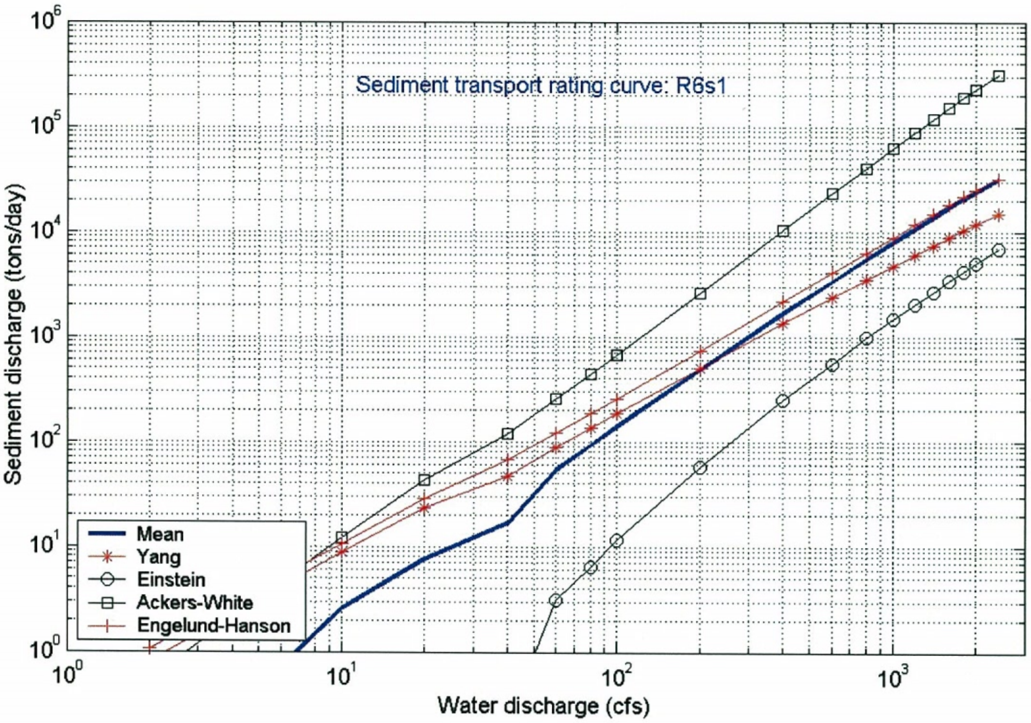


Figure 5-29. Sediment Transport Rating Curve for Sub-Reach R6S1

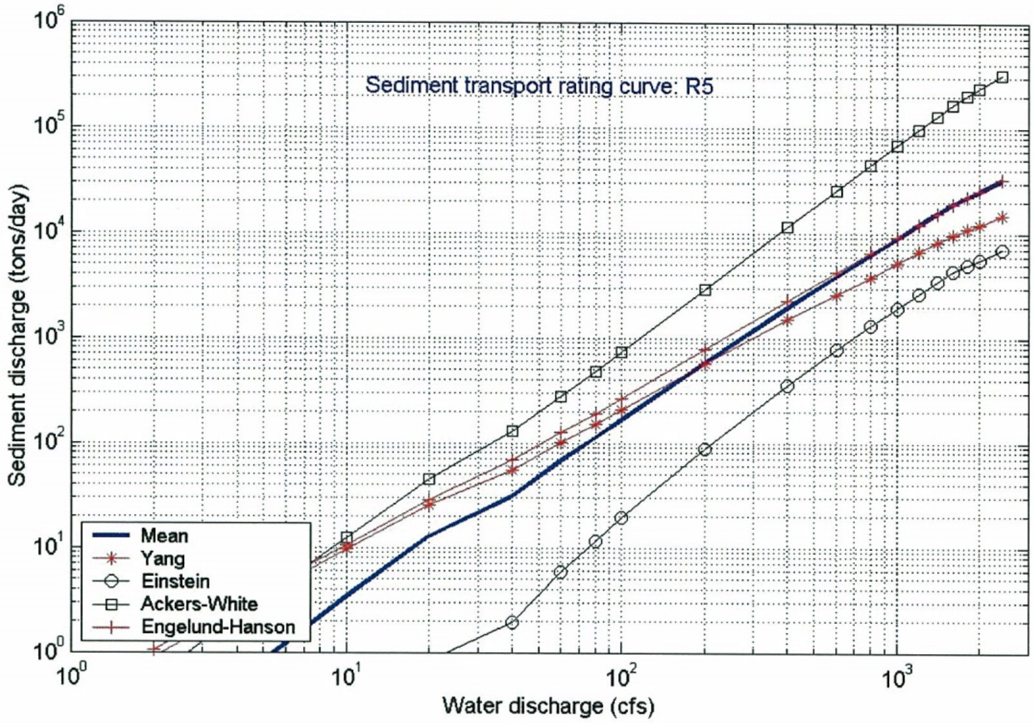


Figure 5-30. Sediment Transport Rating Curve for Sub-Reach R5

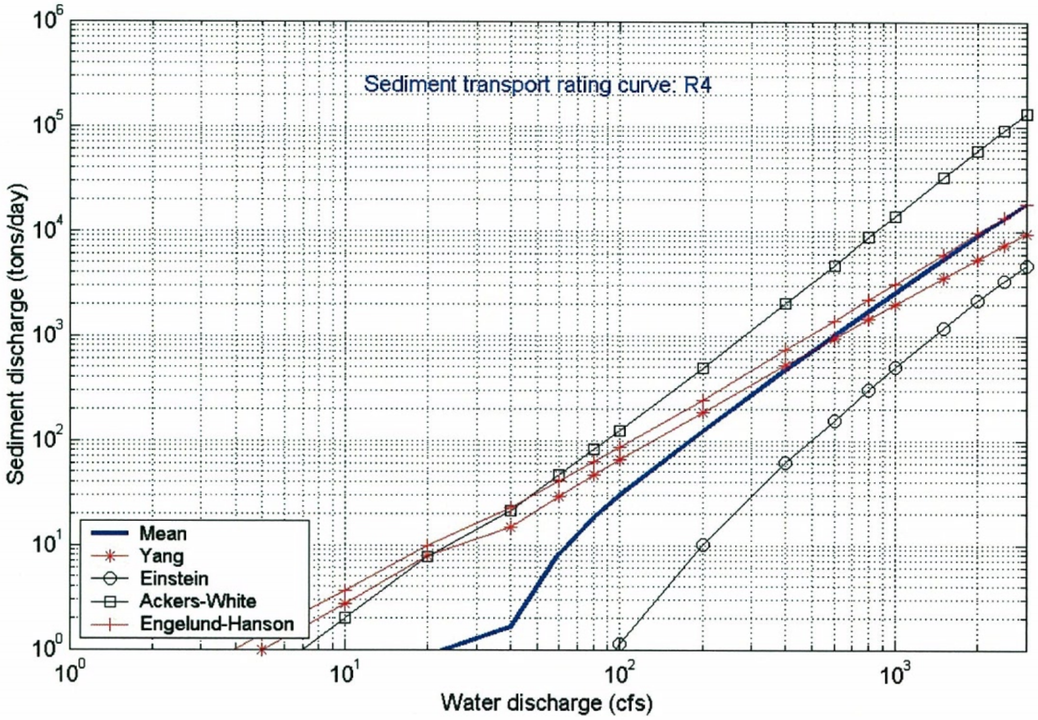


Figure 5-31. Sediment Transport Rating Curve for Sub-Reach R4

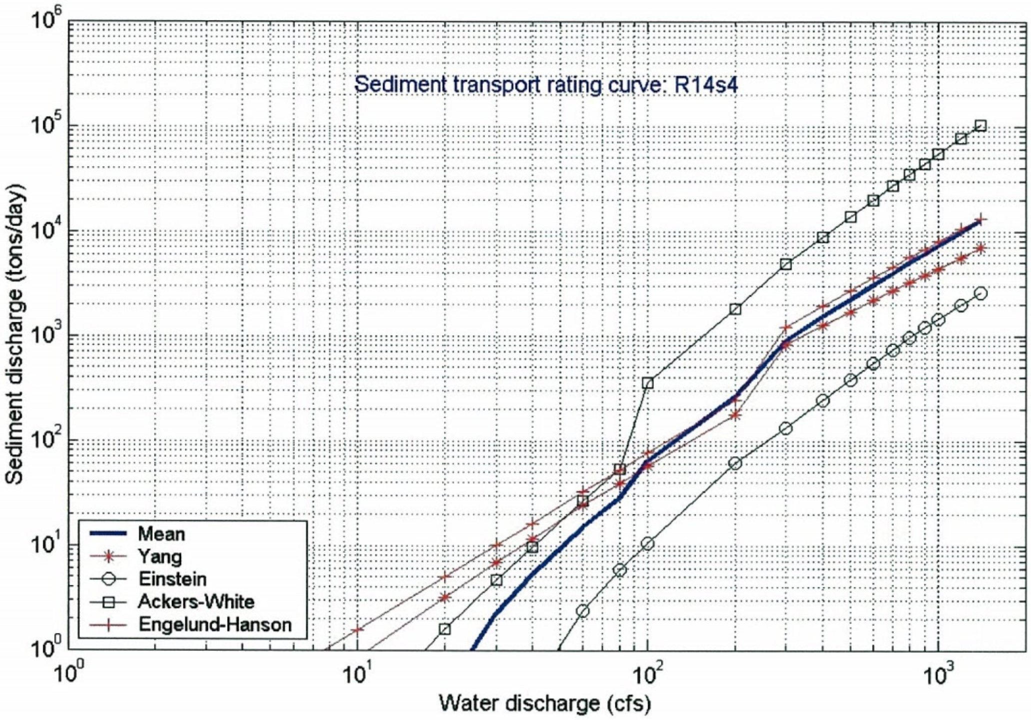


Figure 5-32. Sediment Transport Rating Curve for Sub-Reach R14S4

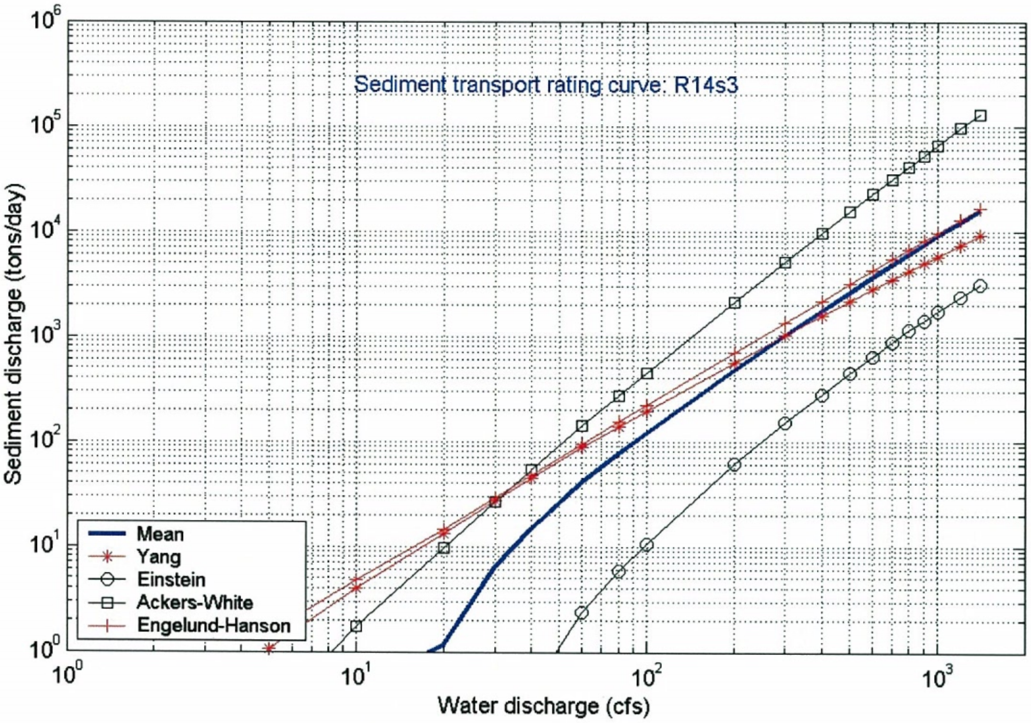


Figure 5-33. Sediment Transport Rating Curve for Sub-Reach R14S3

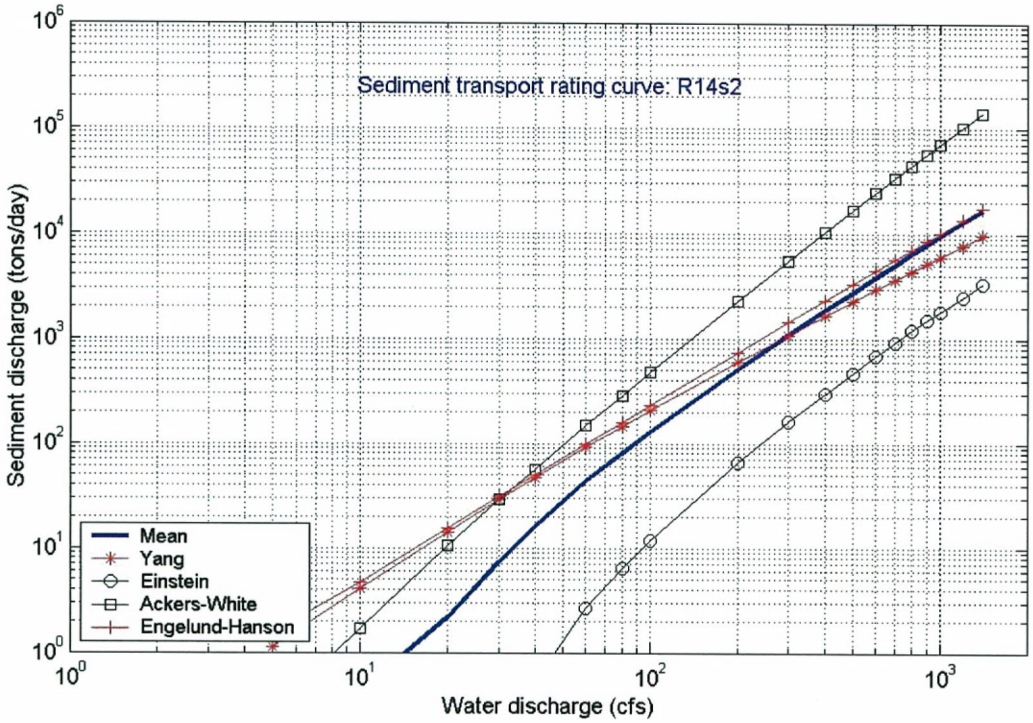


Figure 5-34. Sediment Transport Rating Curve for Sub-Reach R14S2

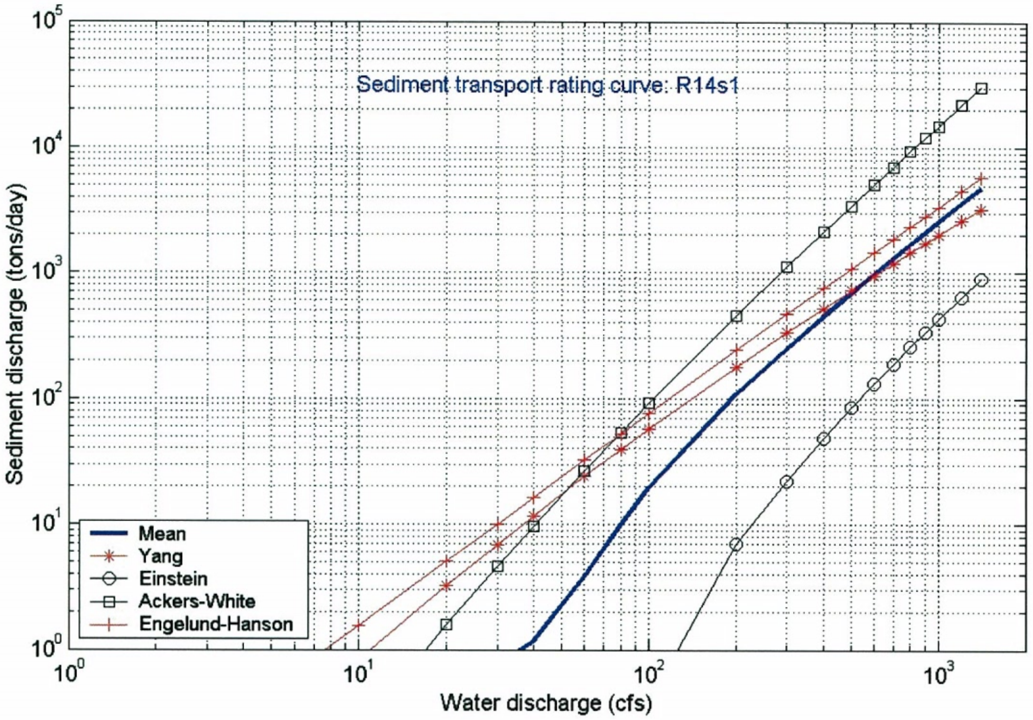
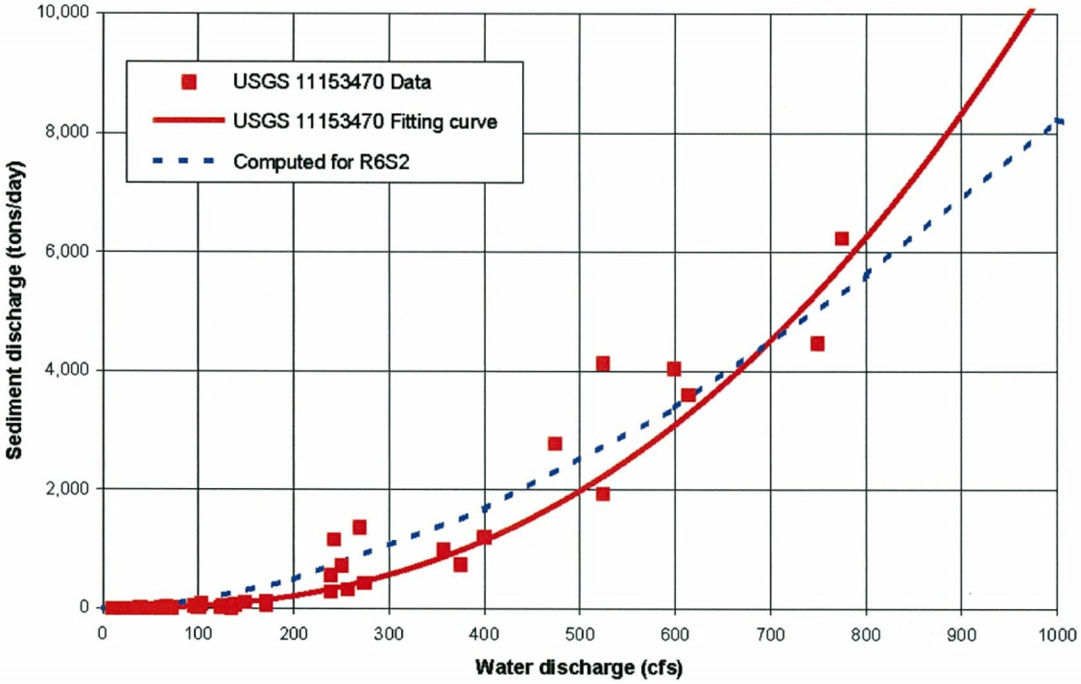


Figure 5-35. Sediment Transport Rating Curve for Sub-Reach R14S1



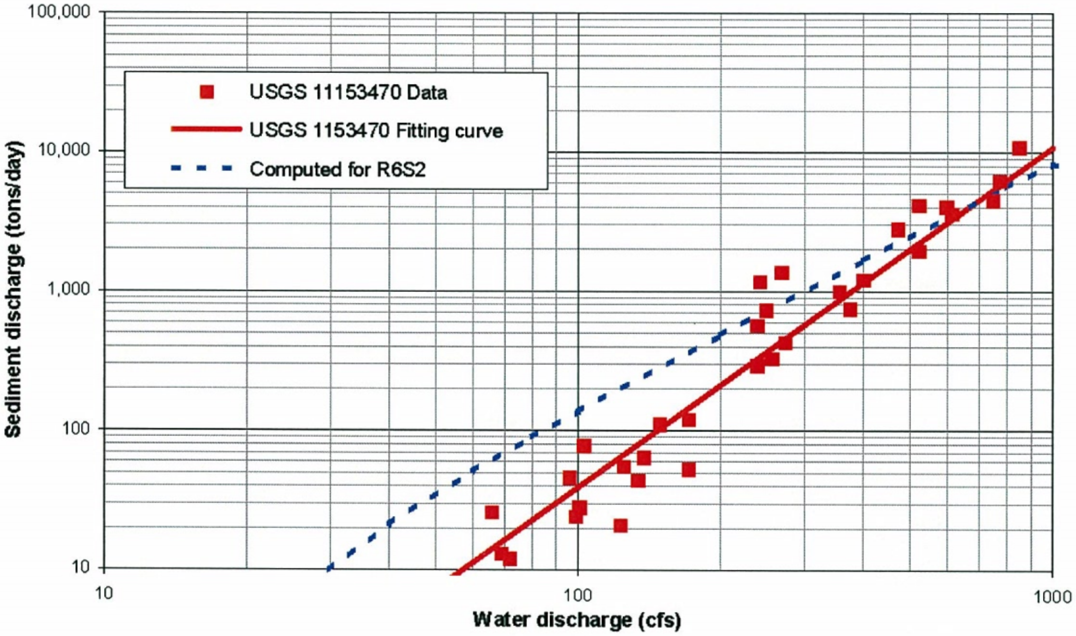


Figure 5-36. Comparison of Sediment Transport Rating Curves between Data at USGS Gage 11153470 and Computation for R6S2

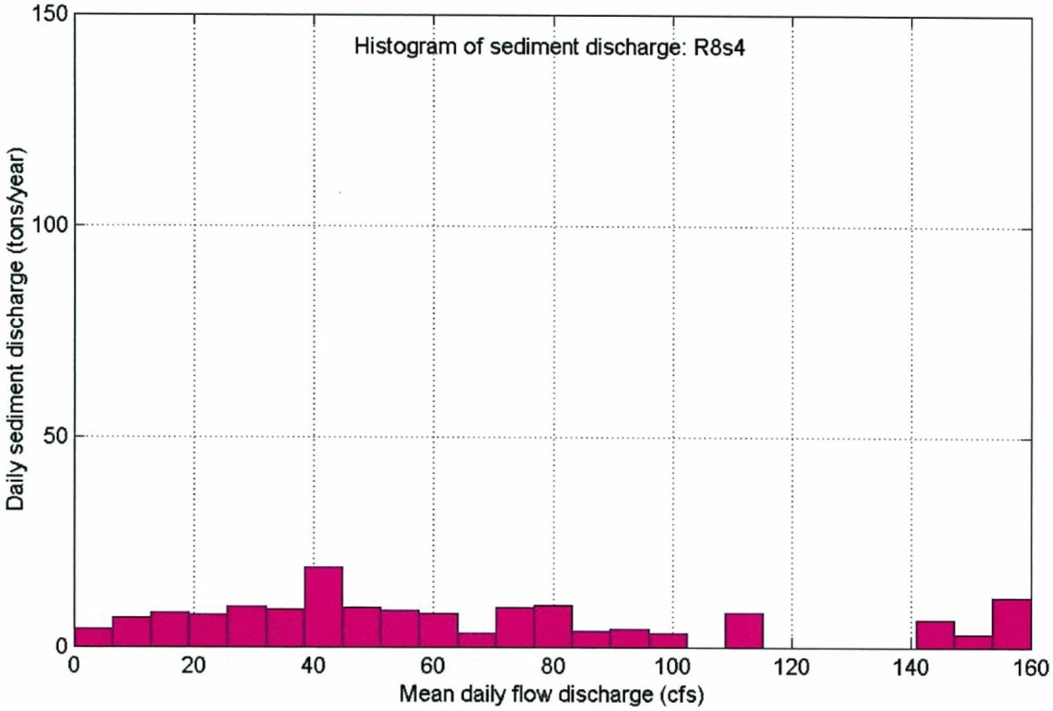


Figure 5-37. Calculated Sediment Discharge Histogram for Sub-reach R8S4

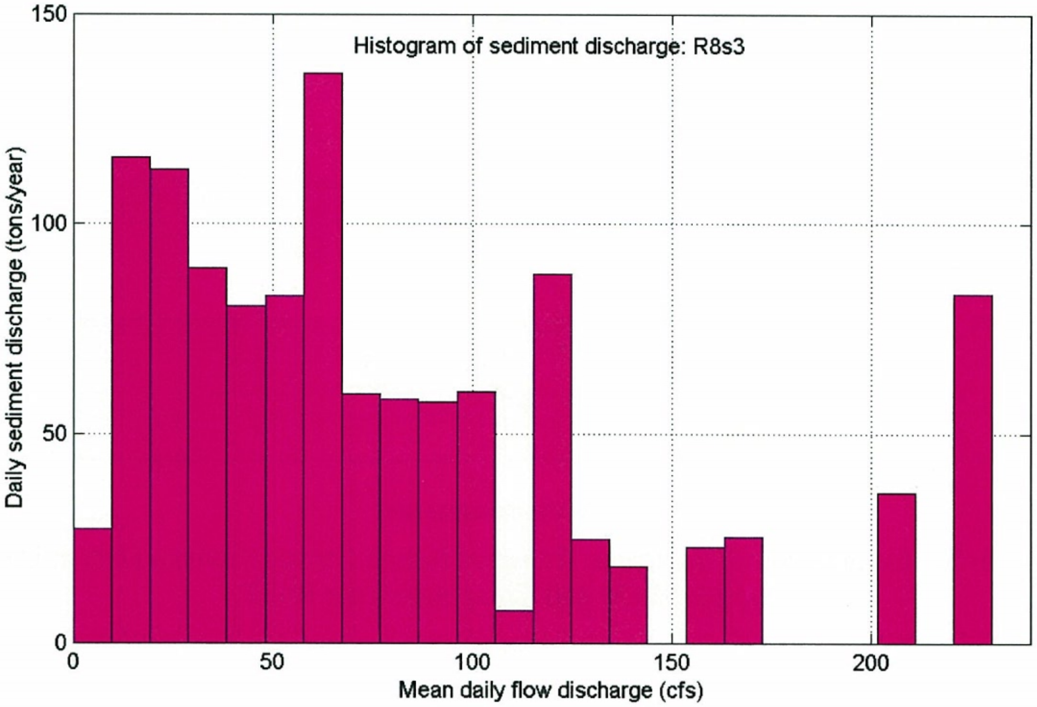


Figure 5-38. Calculated Sediment Discharge Histogram for Sub-reach R8S3

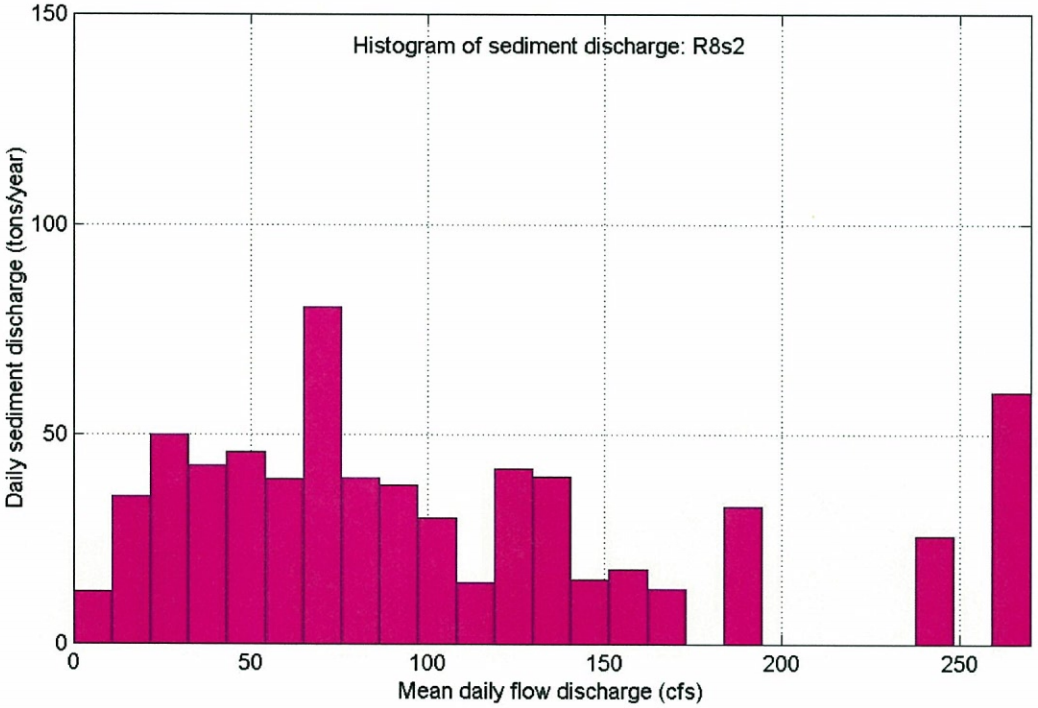


Figure 5-39. Calculated Sediment Discharge Histogram for Sub-reach R8S2

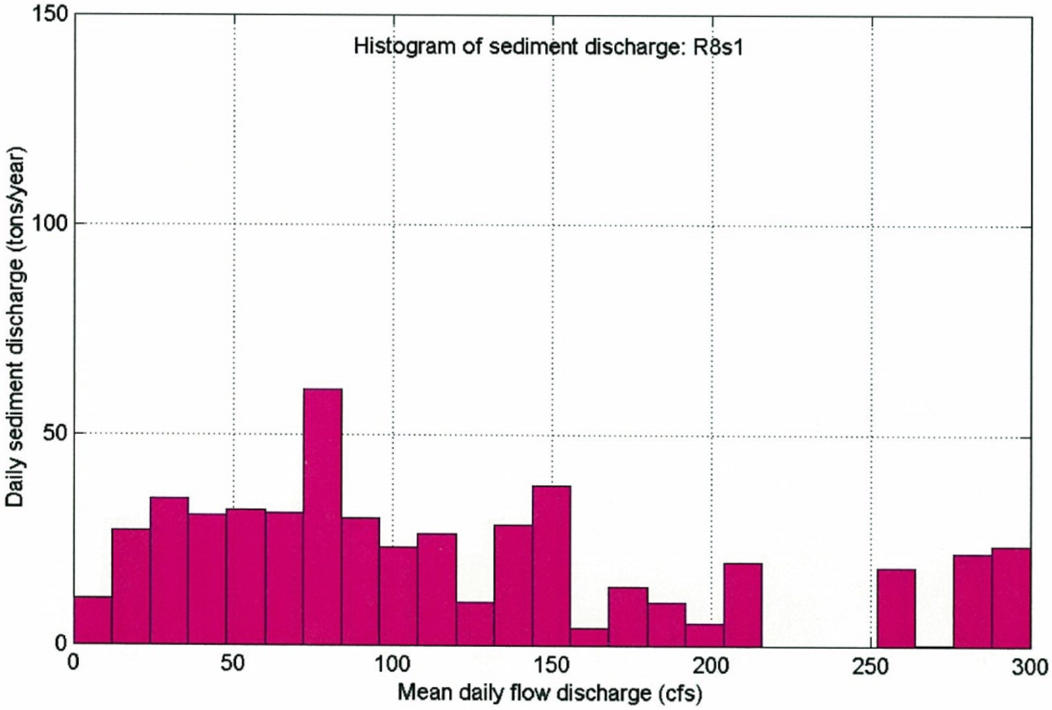


Figure 5-40. Calculated Sediment Discharge Histogram for Sub-reach R8S1

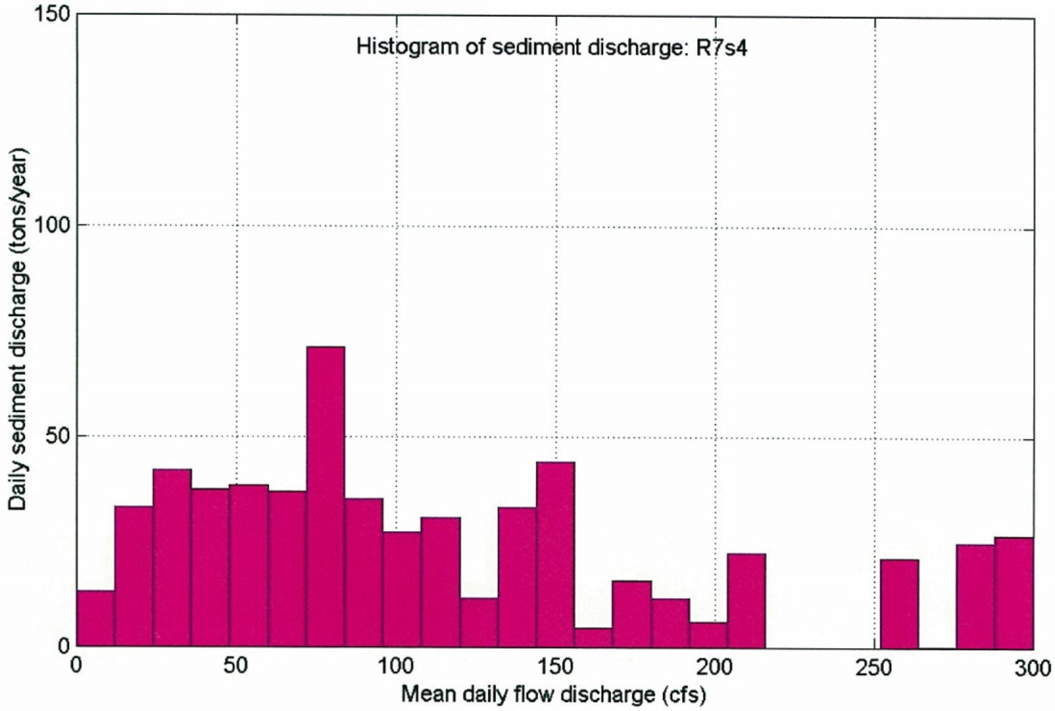


Figure 5-41. Calculated Sediment Discharge Histogram for Sub-reach R7S4

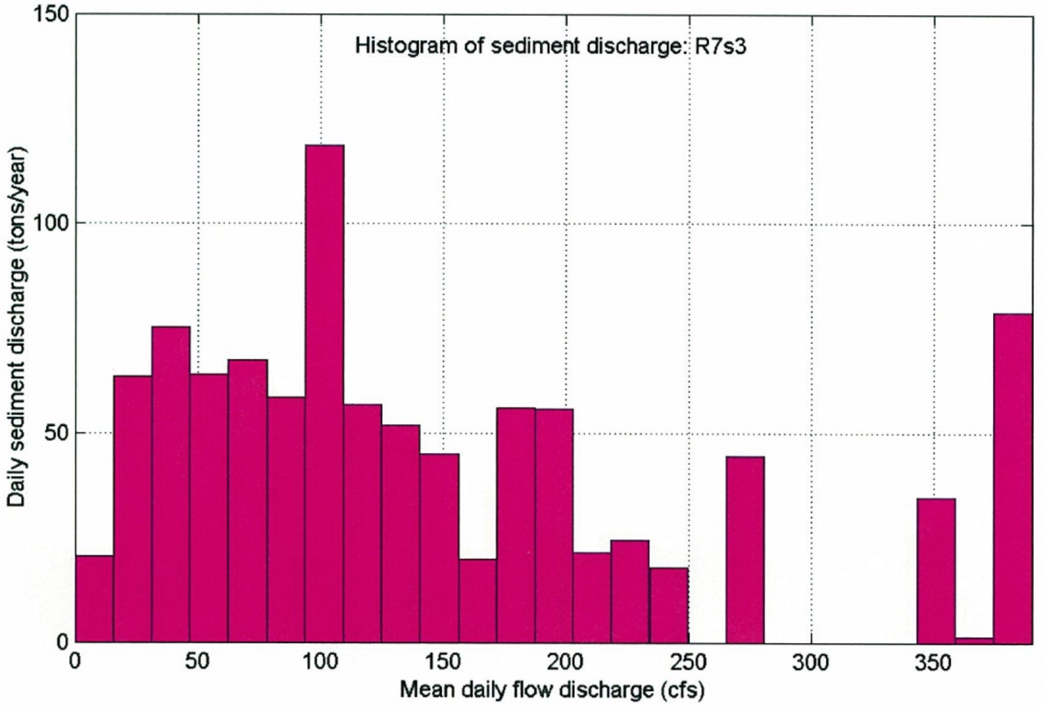


Figure 5-42. Calculated Sediment Discharge Histogram for Sub-reach R7S3

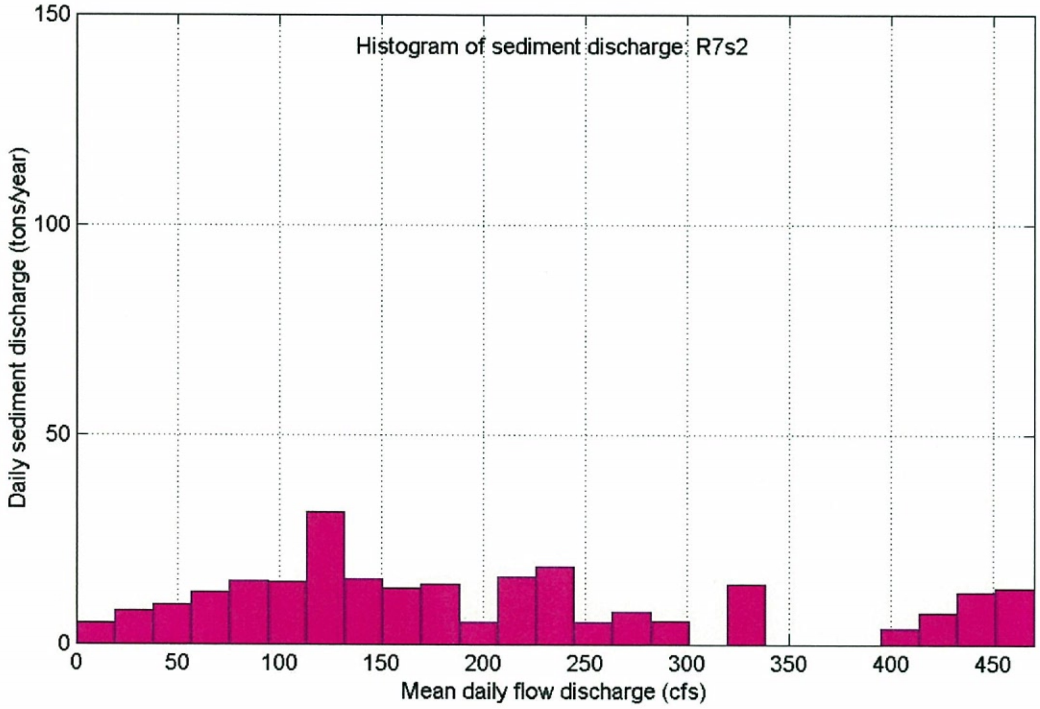


Figure 5-43. Calculated Sediment Discharge Histogram for Sub-reach R7S2

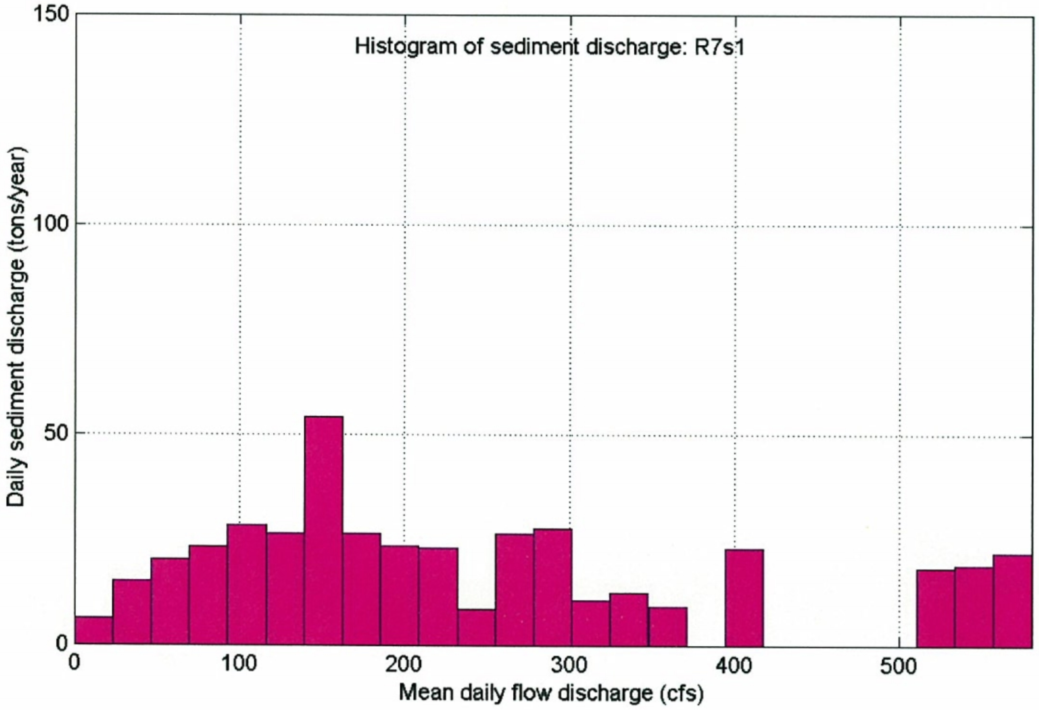


Figure 5-44. Calculated Sediment Discharge Histogram for Sub-reach R7S1

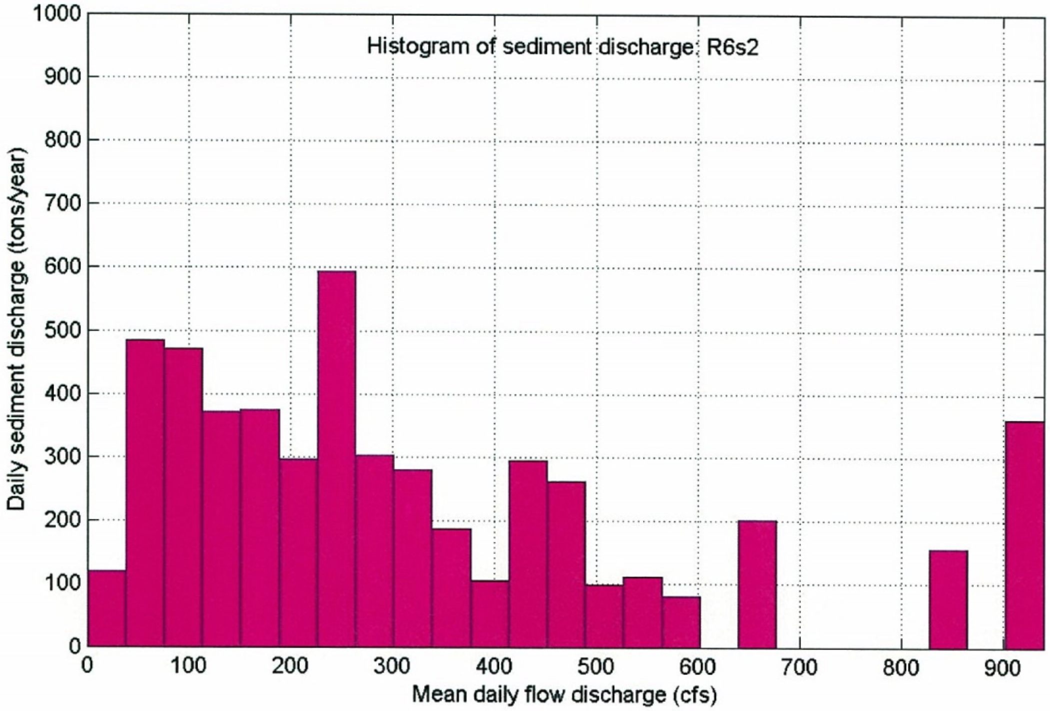


Figure 5-45. Calculated Sediment Discharge Histogram for Sub-reach R6S2

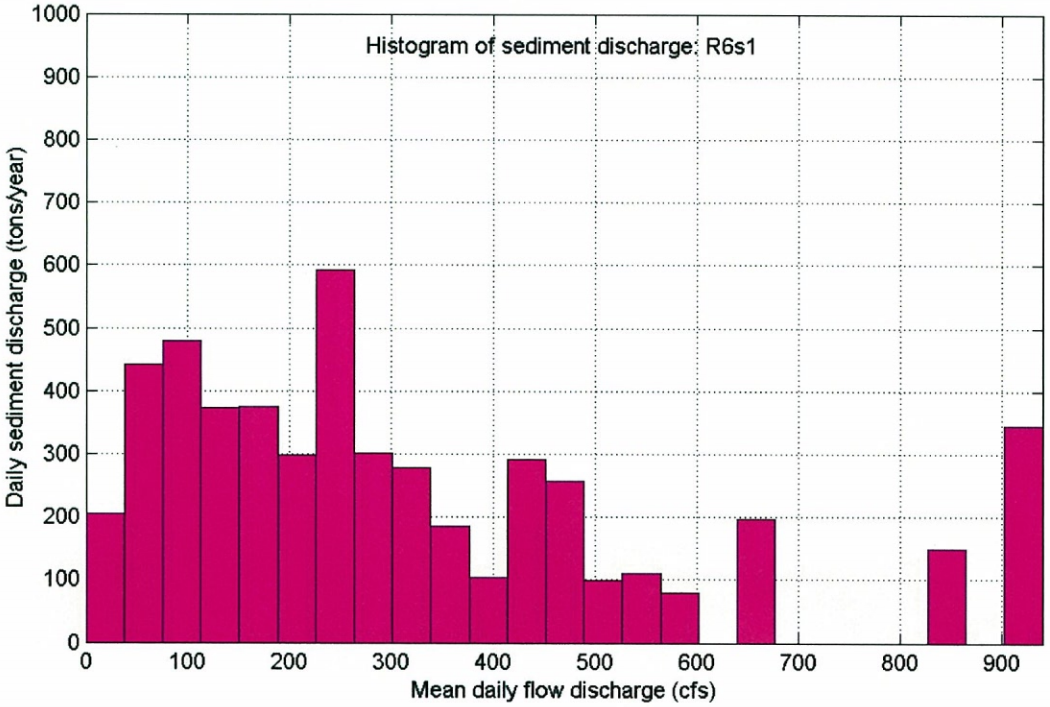


Figure 5-46. Calculated Sediment Discharge Histogram for Sub-reach R6S1

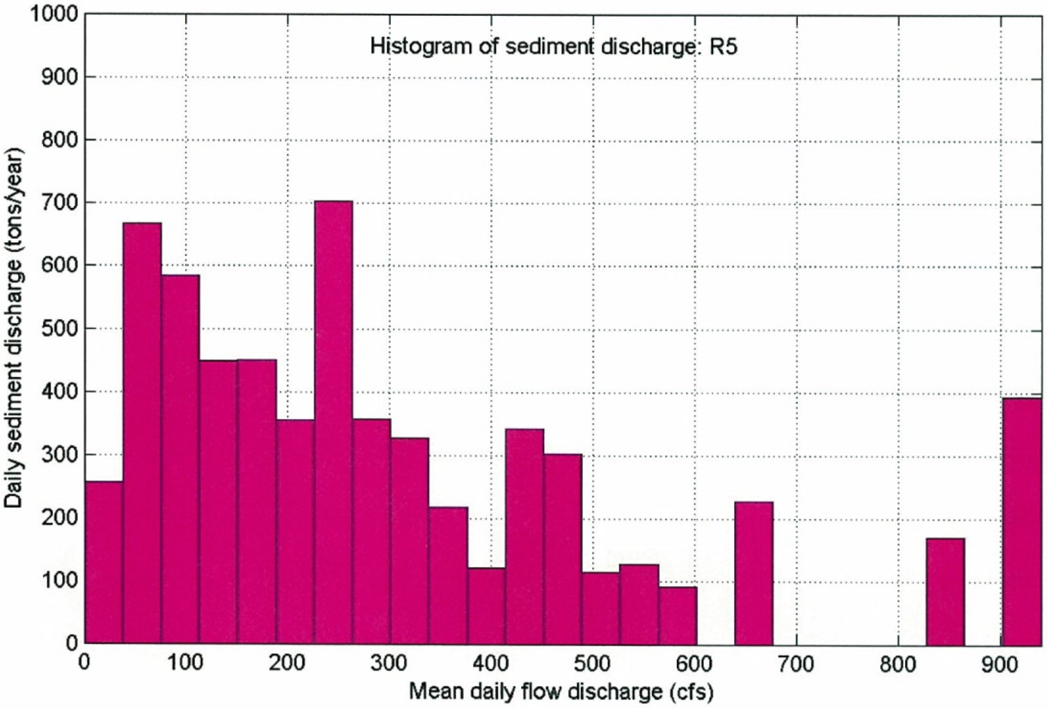


Figure 5-47. Calculated Sediment Discharge Histogram for Reach R5

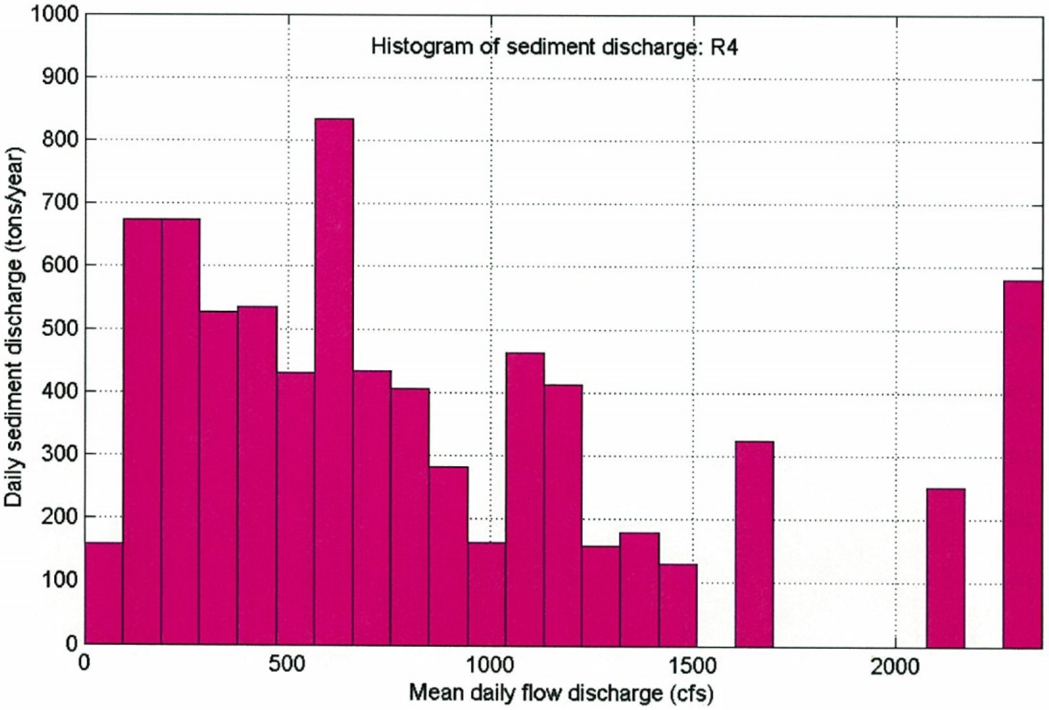


Figure 5-48. Calculated Sediment Discharge Histogram for Reach R4

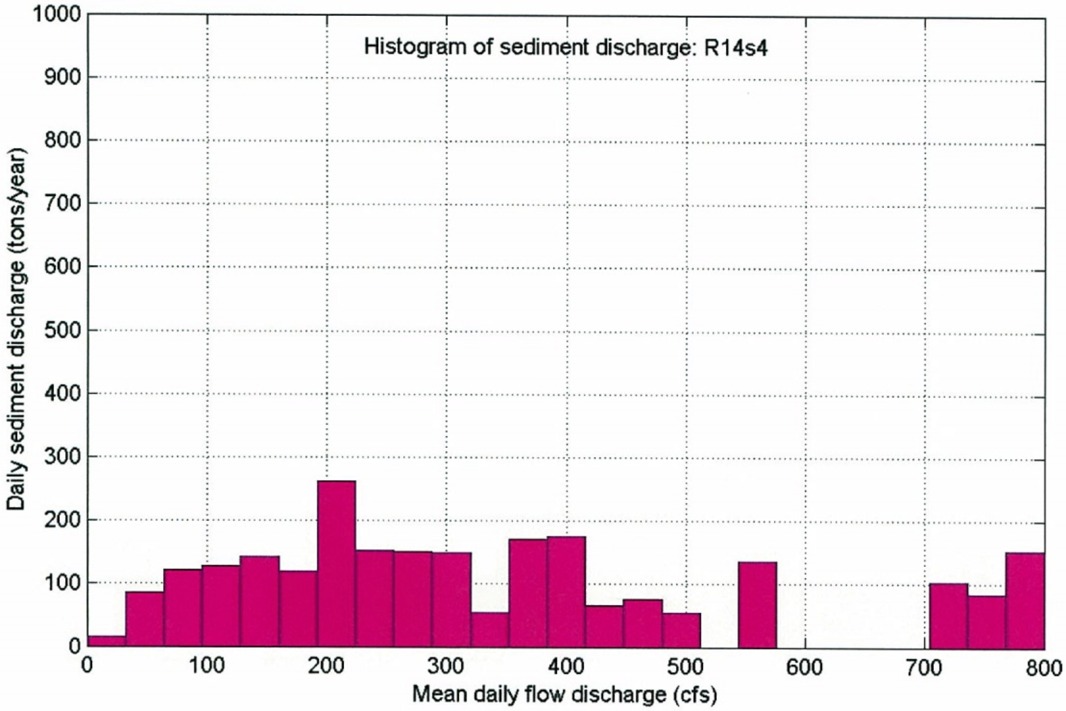


Figure 5-49. Calculated Sediment Discharge Histogram for Sub-reach R14S4

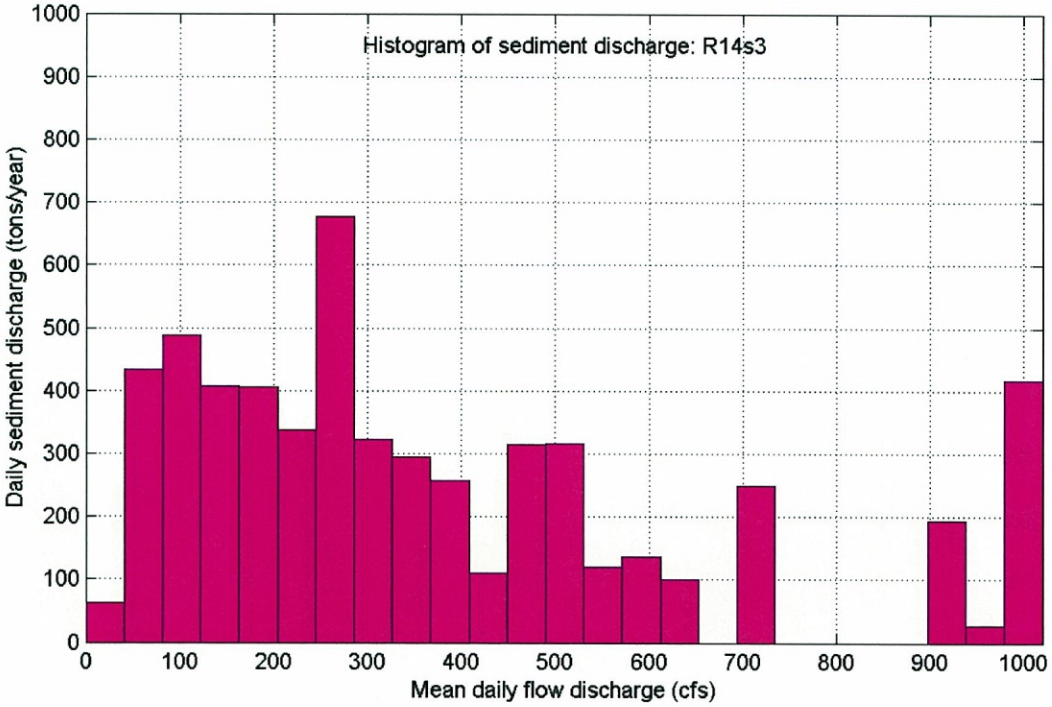


Figure 5-50. Calculated Sediment Discharge Histogram for Sub-reach R14S3

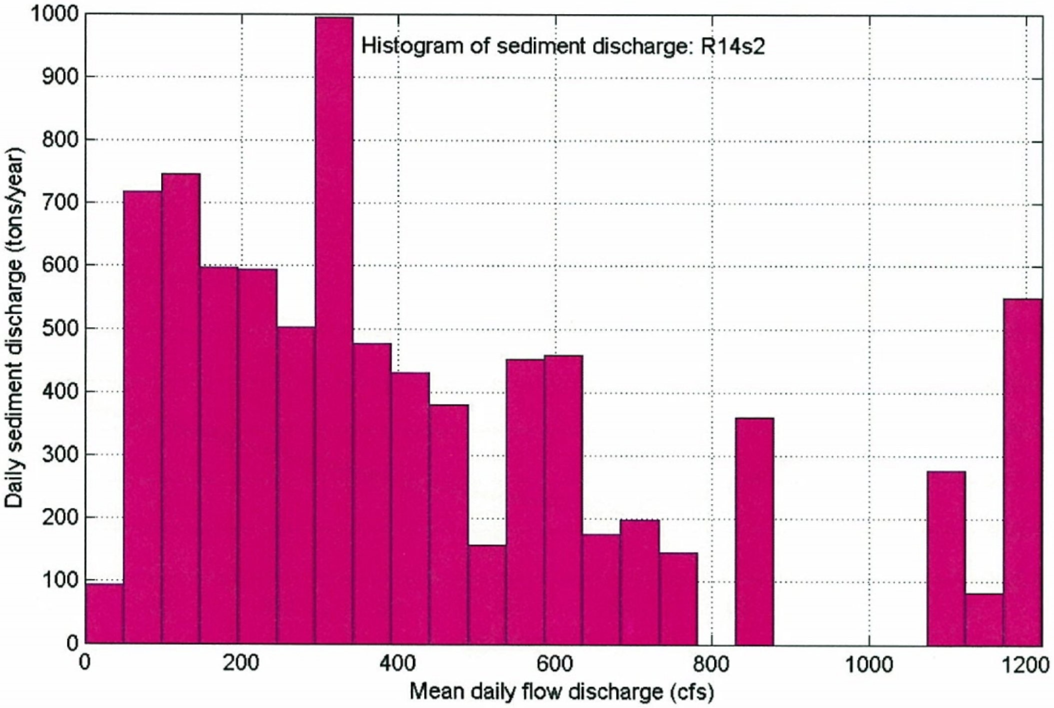


Figure 5-51. Calculated Sediment Discharge Histogram for Sub-reach R14S2

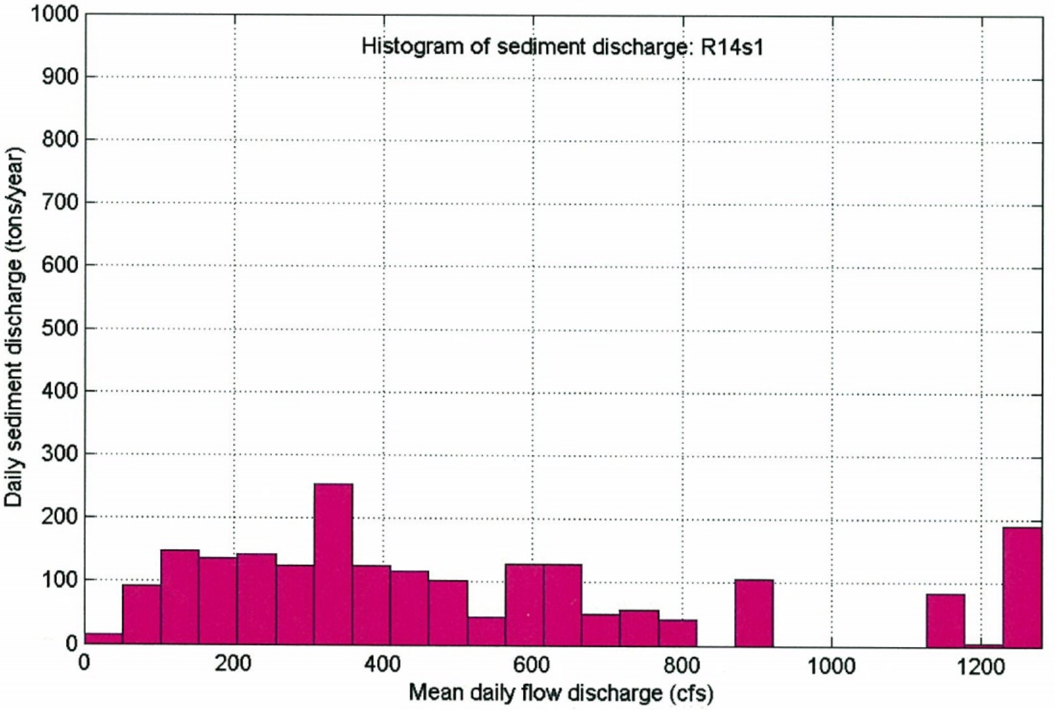


Figure 5-52. Calculated Sediment Discharge Histogram for Sub-reach R14S1

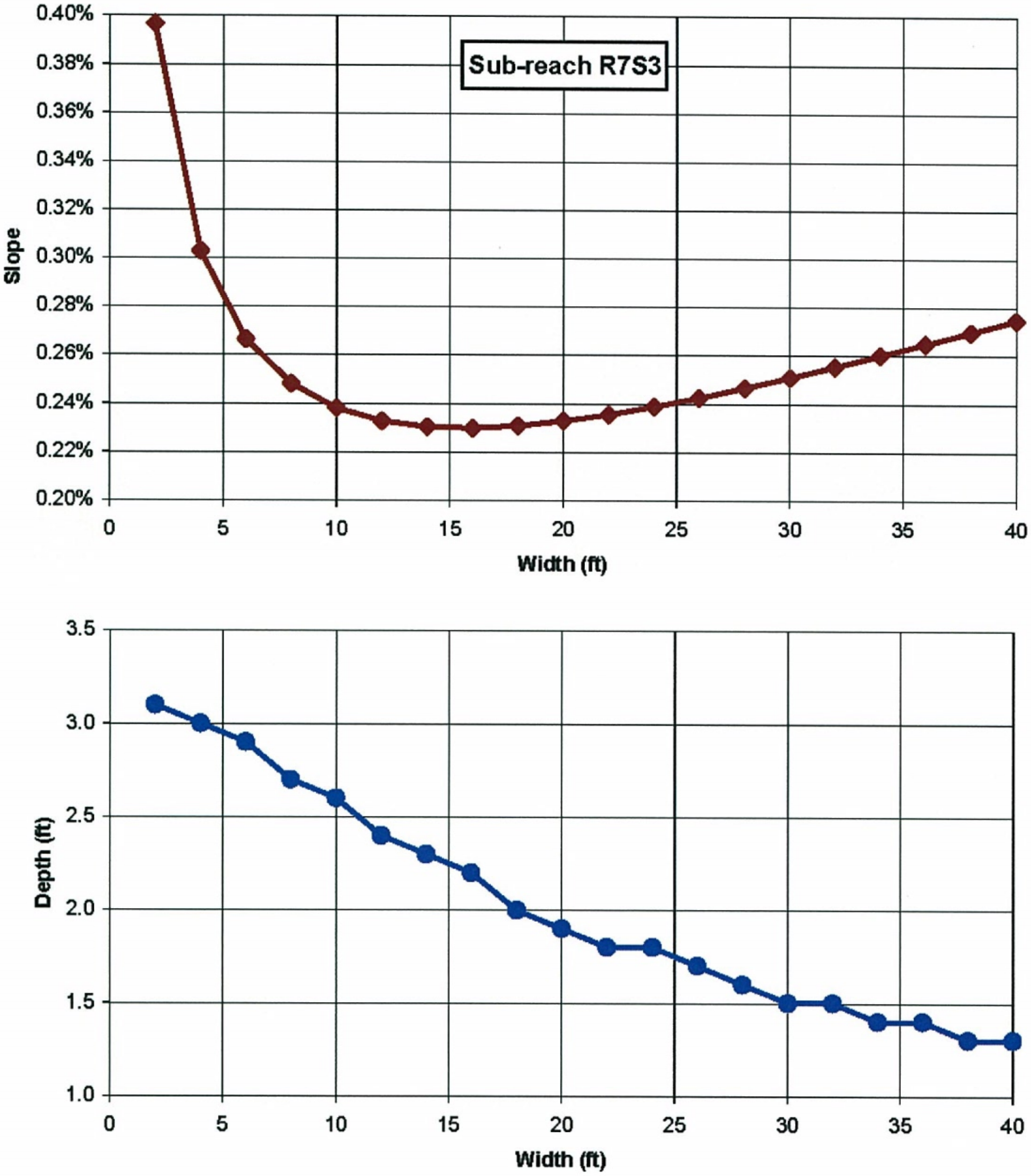


Figure 5-53. Slope-Width and Depth-Width Solutions Calculated by SAM for R7S3

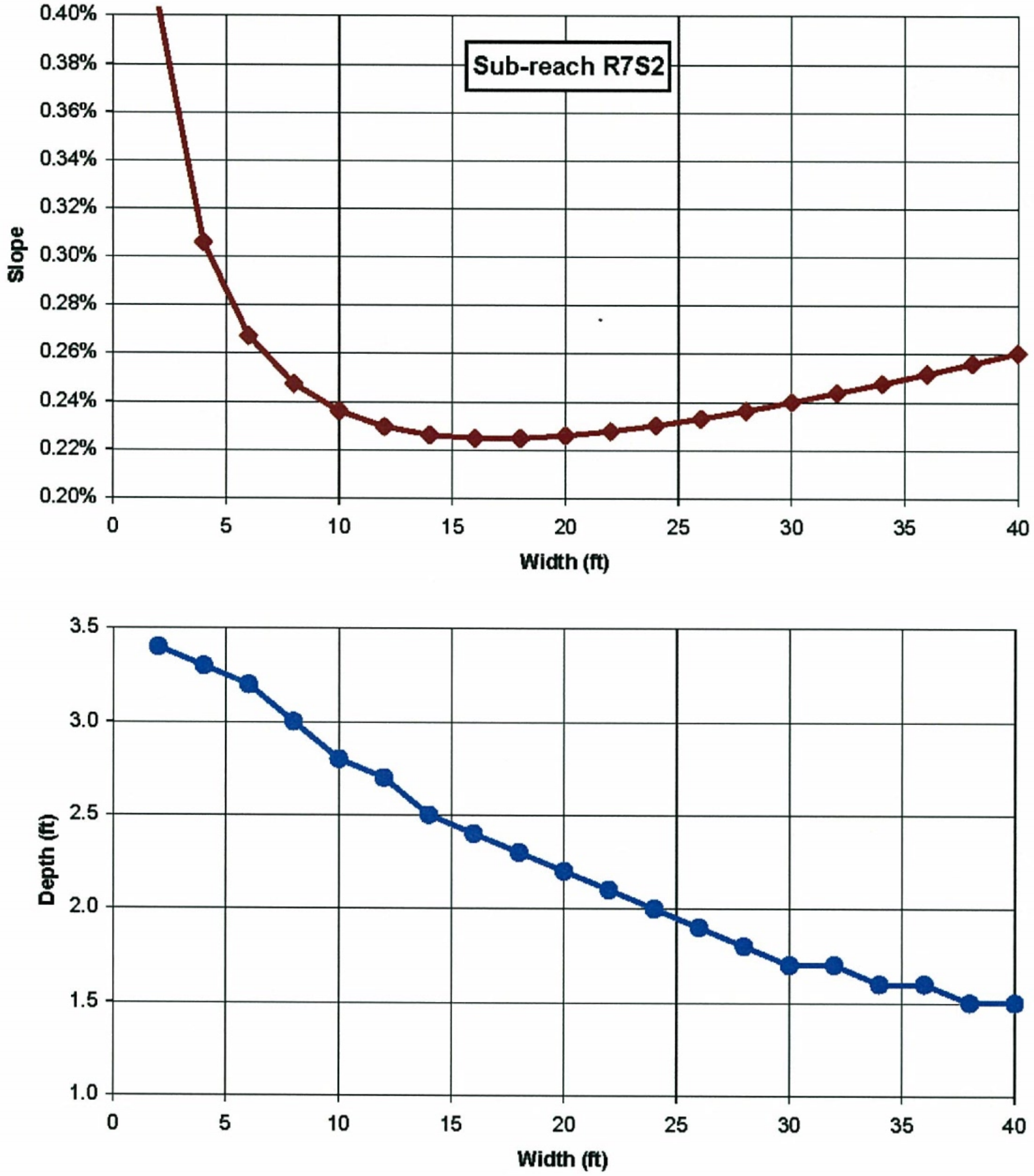


Figure 5-54. Slope-Width and Depth-Width Solutions Calculated by SAM for R7S2

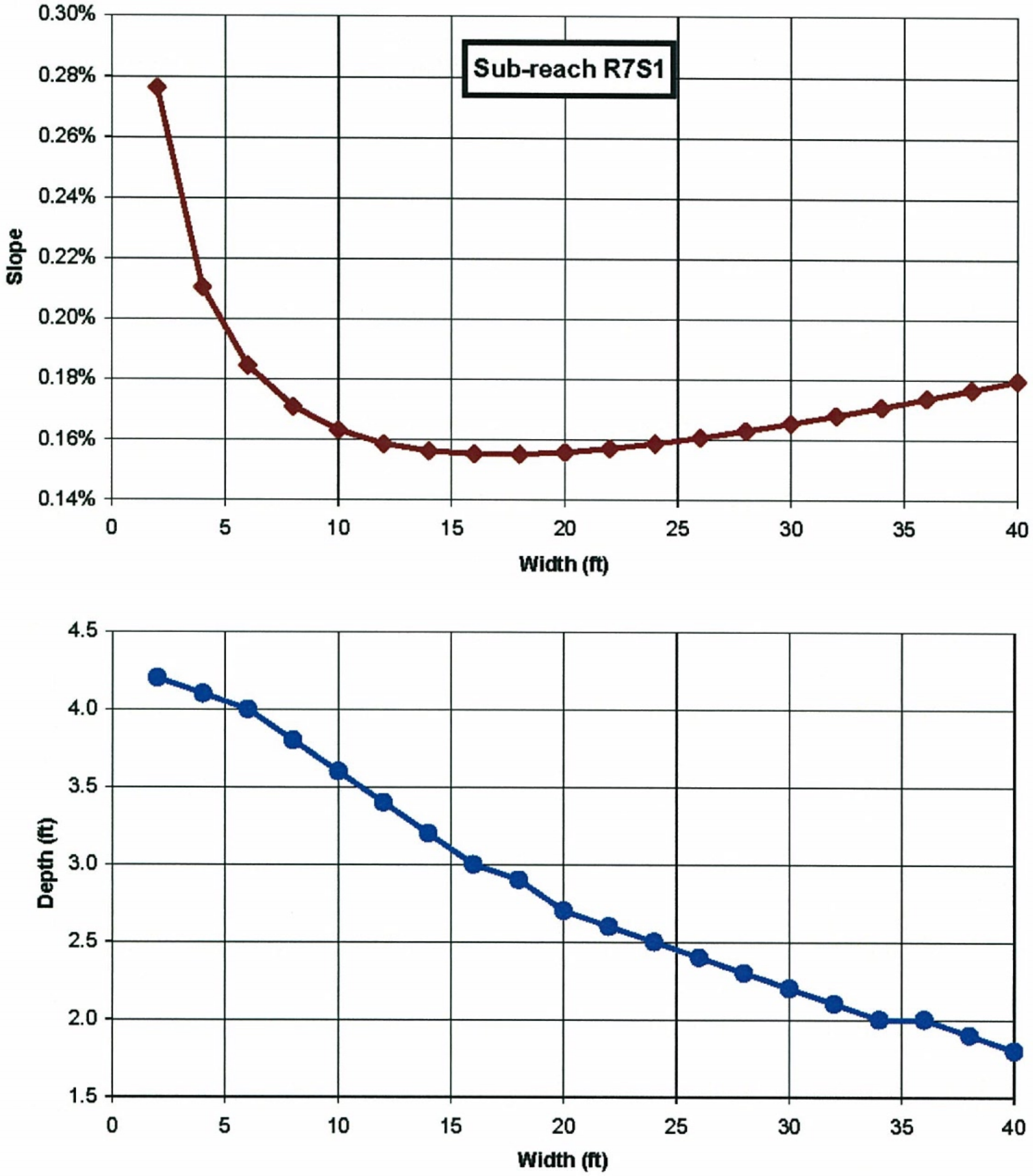


Figure 5-55. Slope-Width and Depth-Width Solutions Calculated by SAM for R7S1

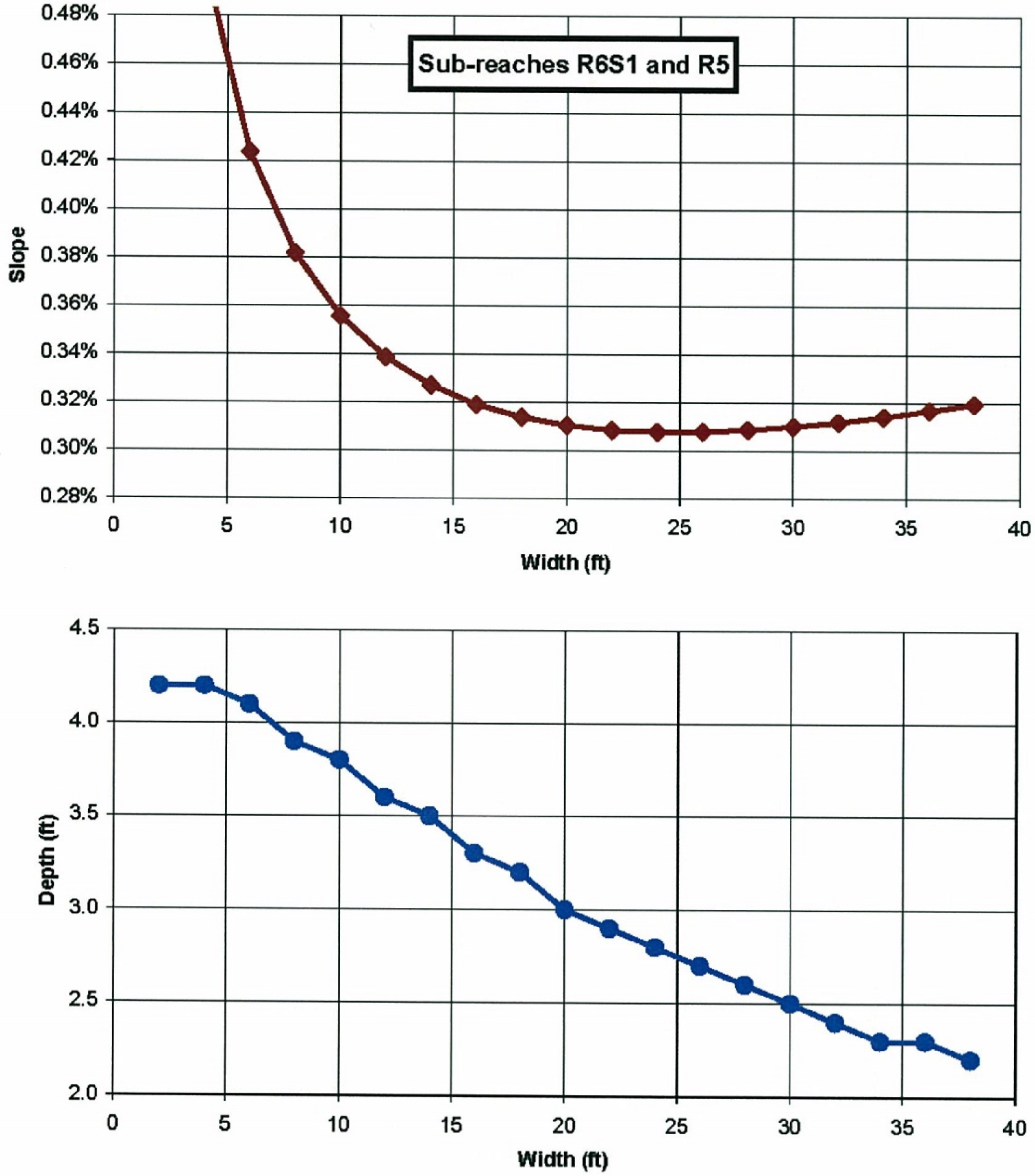


Figure 5-56. Slope-Width and Depth-Width Solutions Calculated by SAM for R6S1 & R5

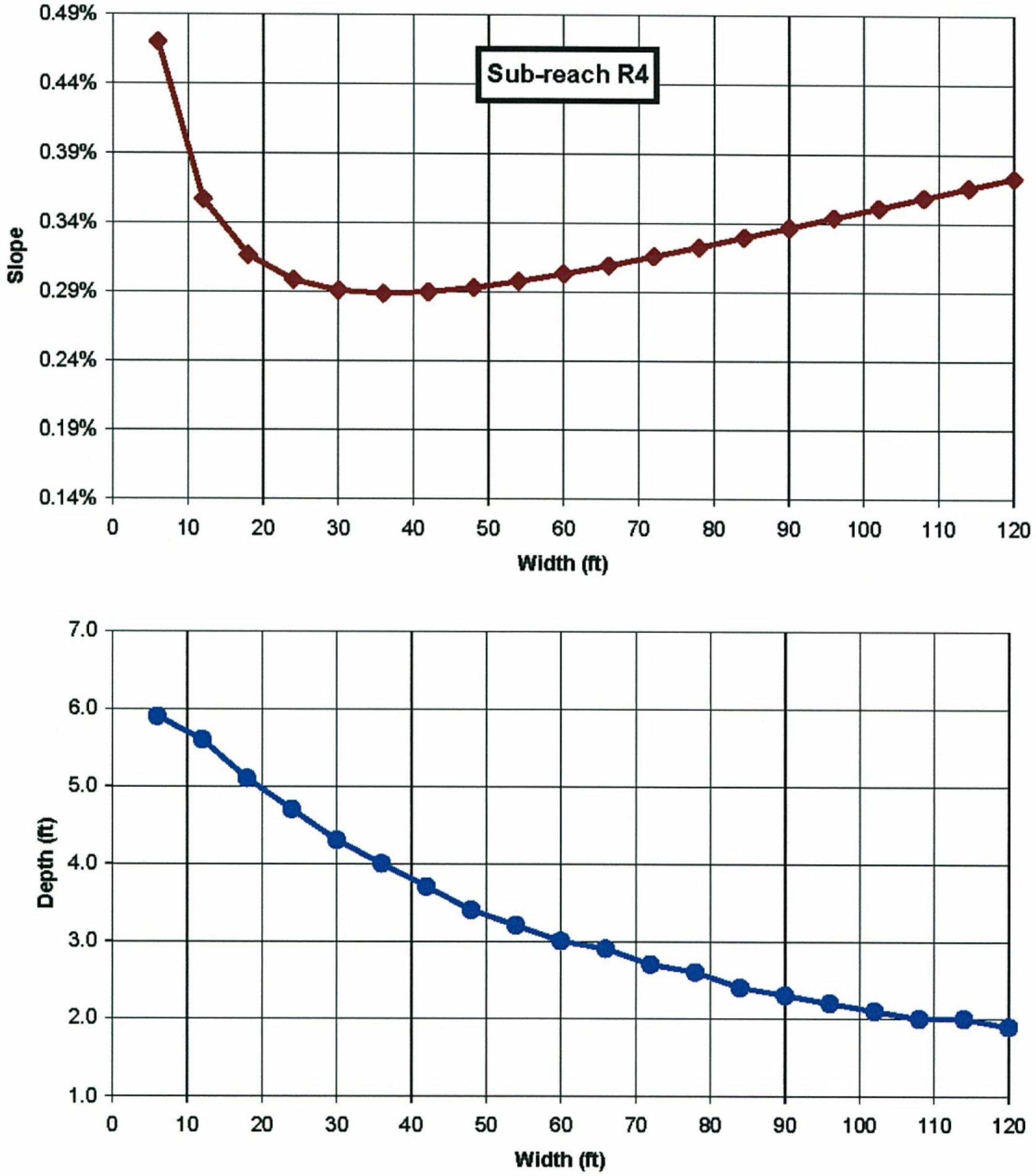


Figure 5-57. Slope-Width and Depth-Width Solutions Calculated by SAM for R4

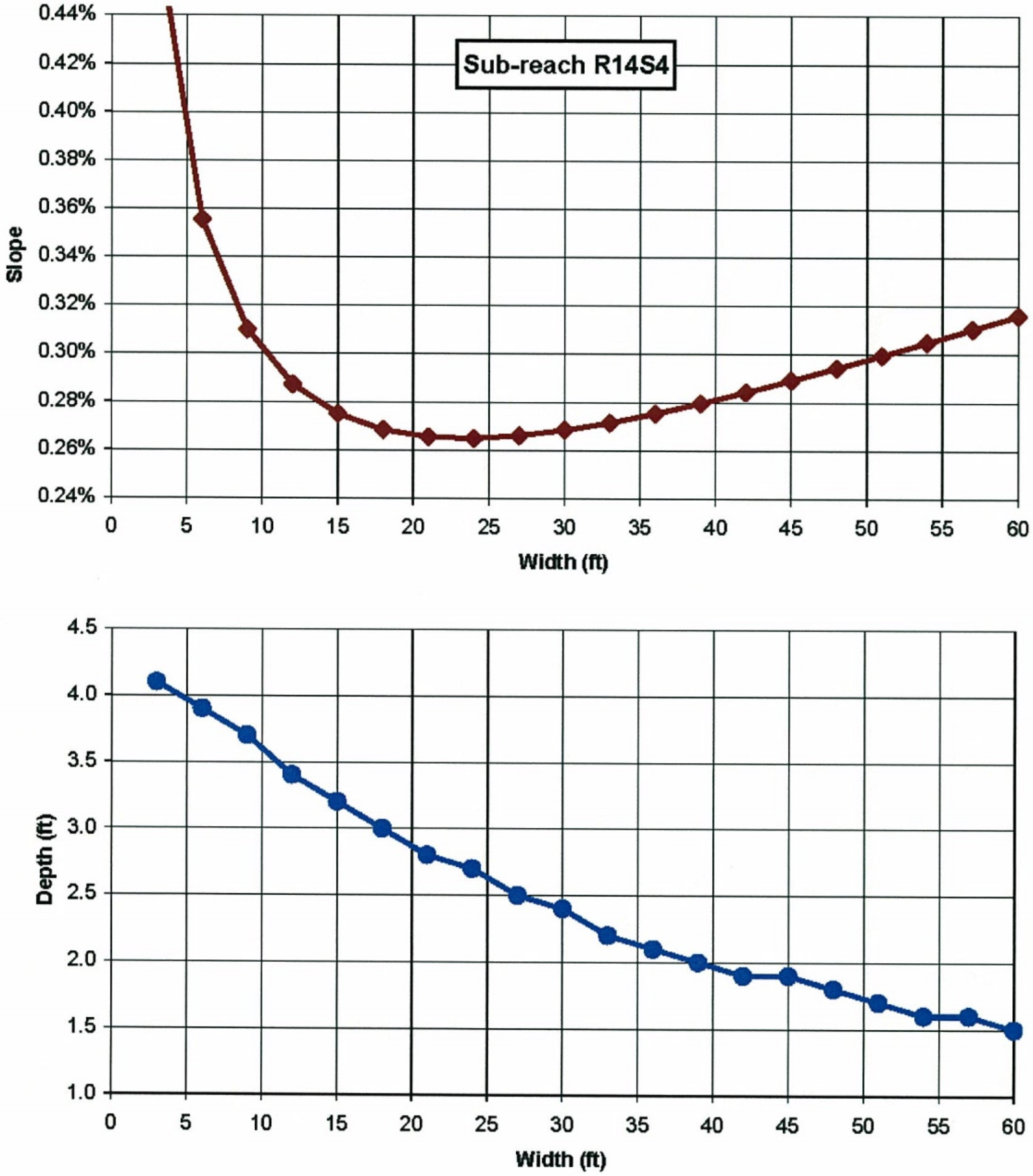


Figure 5-58. Slope-Width and Depth-Width Solutions Calculated by SAM for R14S4

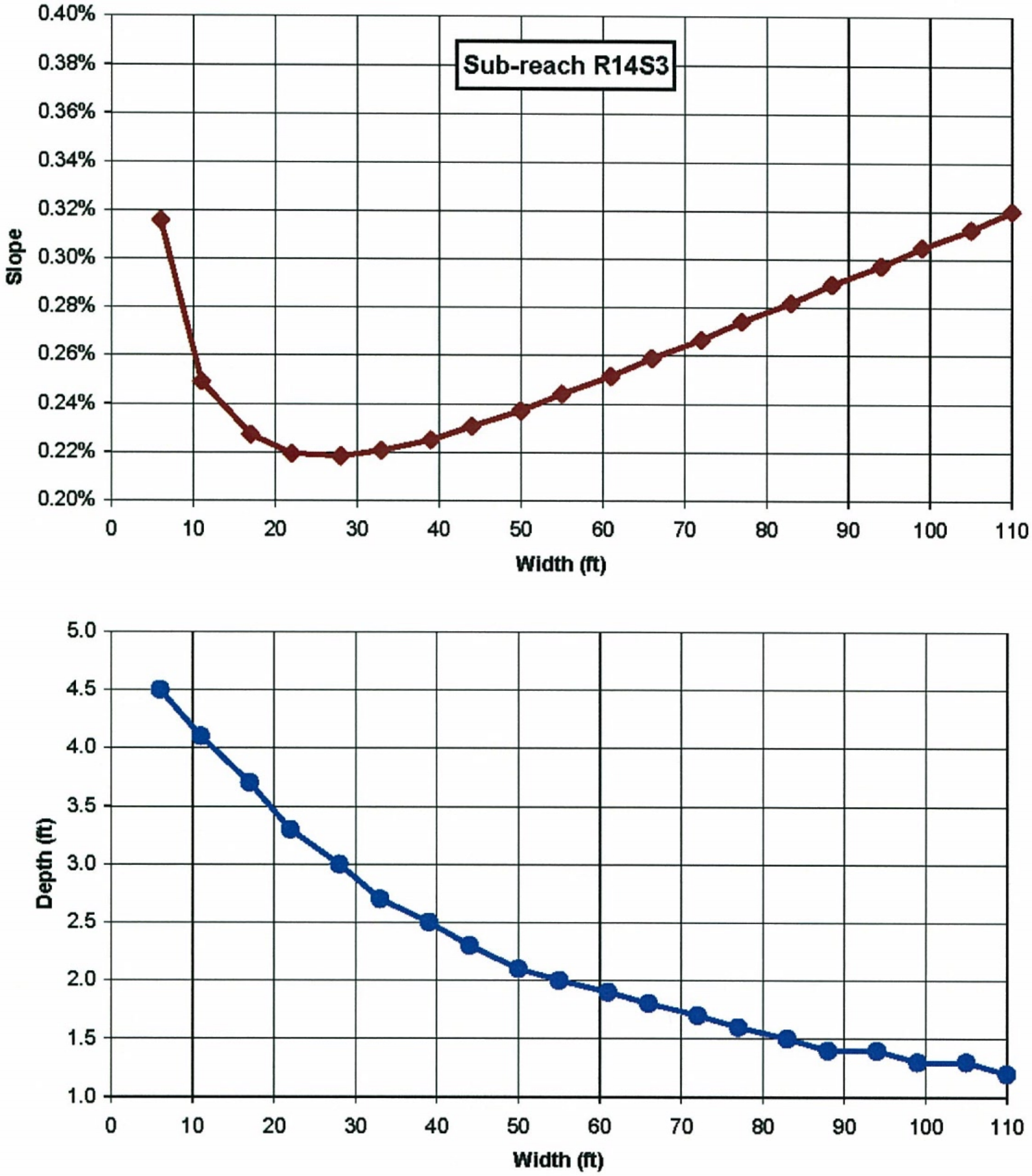


Figure 5-59. Slope-Width and Depth-Width Solutions Calculated by SAM for R14S3

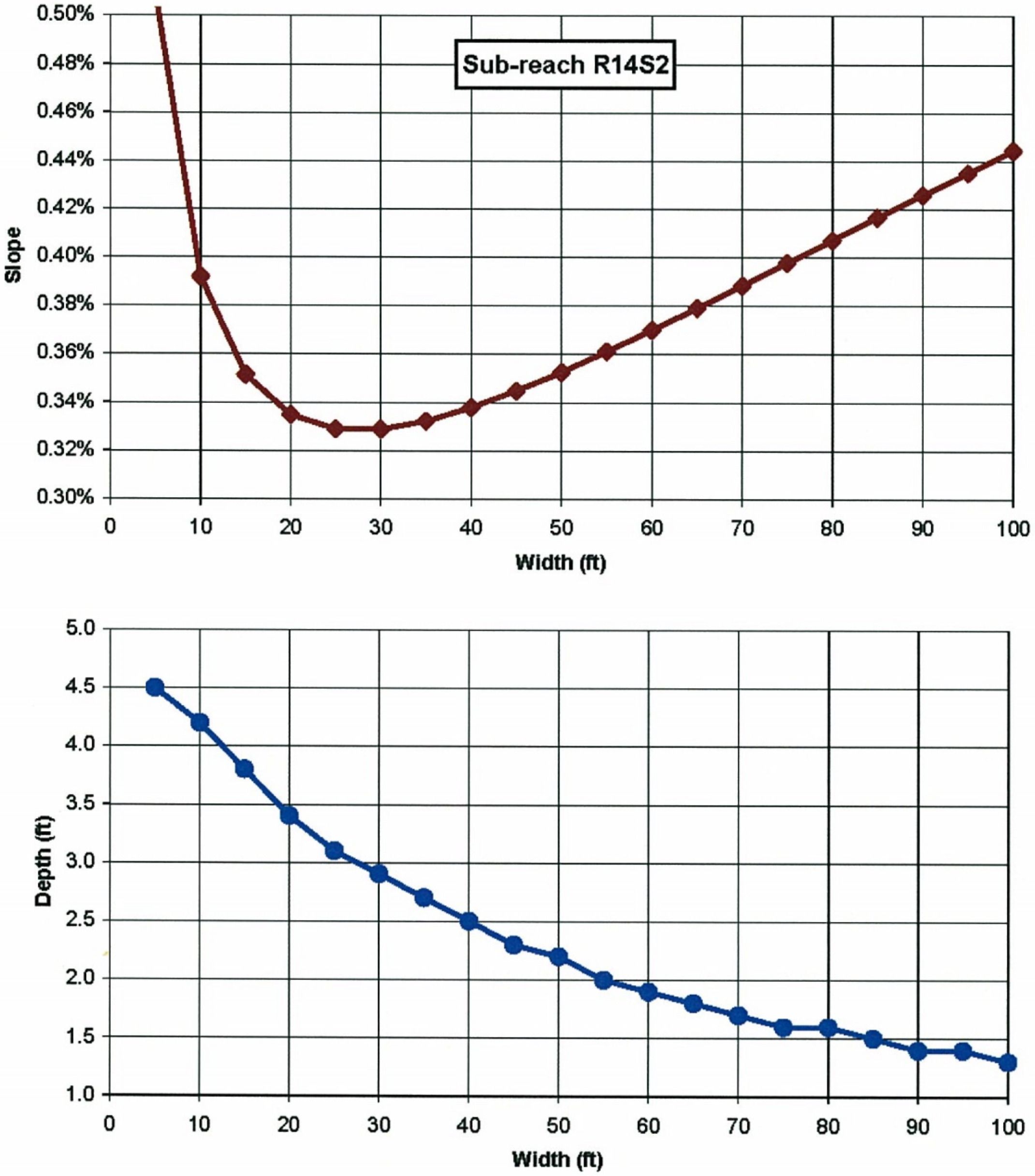


Figure 5-60. Slope-Width and Depth-Width Solutions Calculated by SAM for R14S2

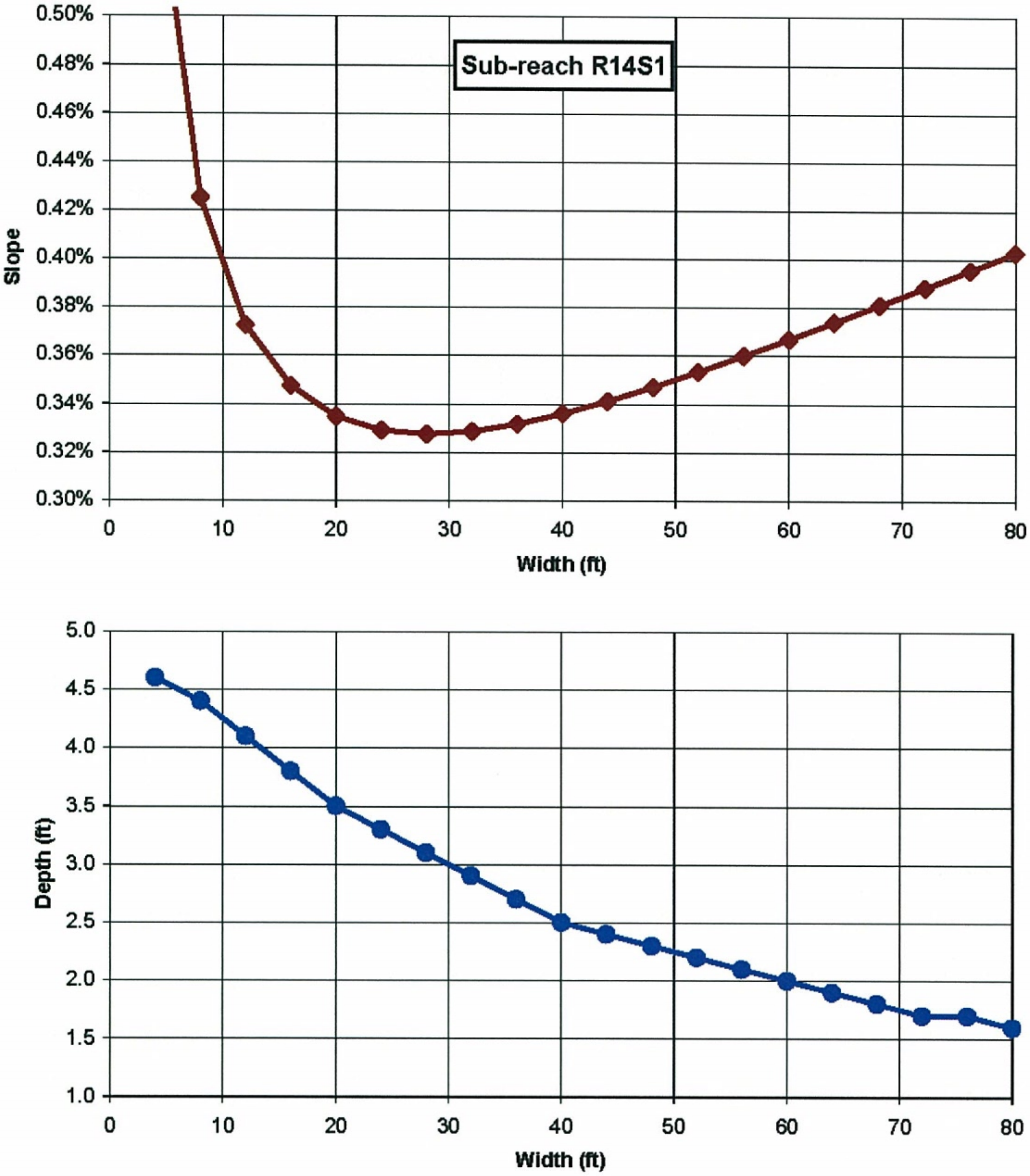


Figure 5-61. Slope-Width and Depth-Width Solutions Calculated by SAM for R14S1

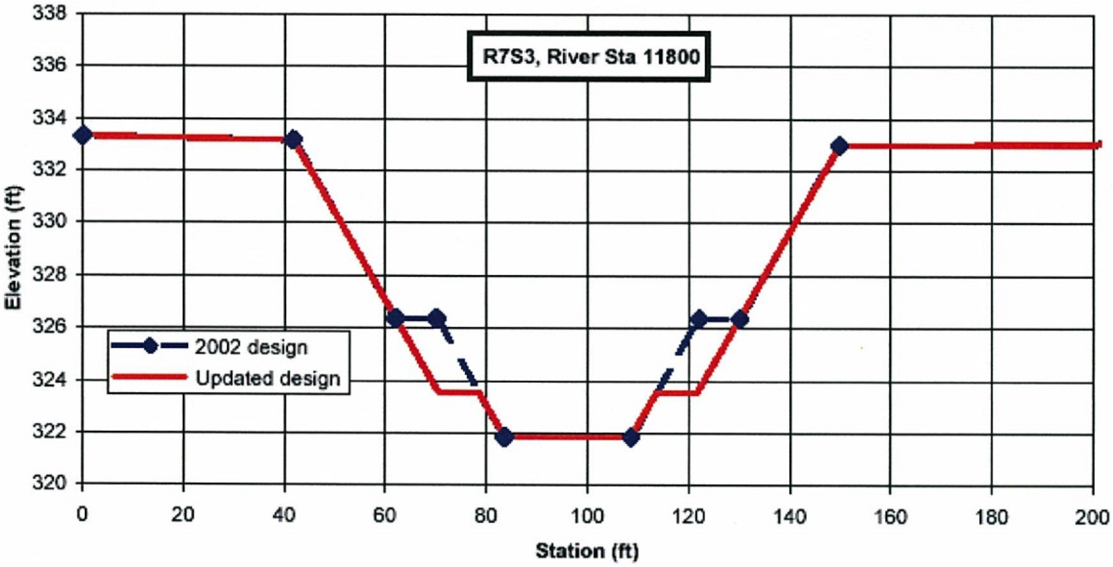


Figure 5-62. Example of Updated Channel Design Compared to 2002 Design for R7S3

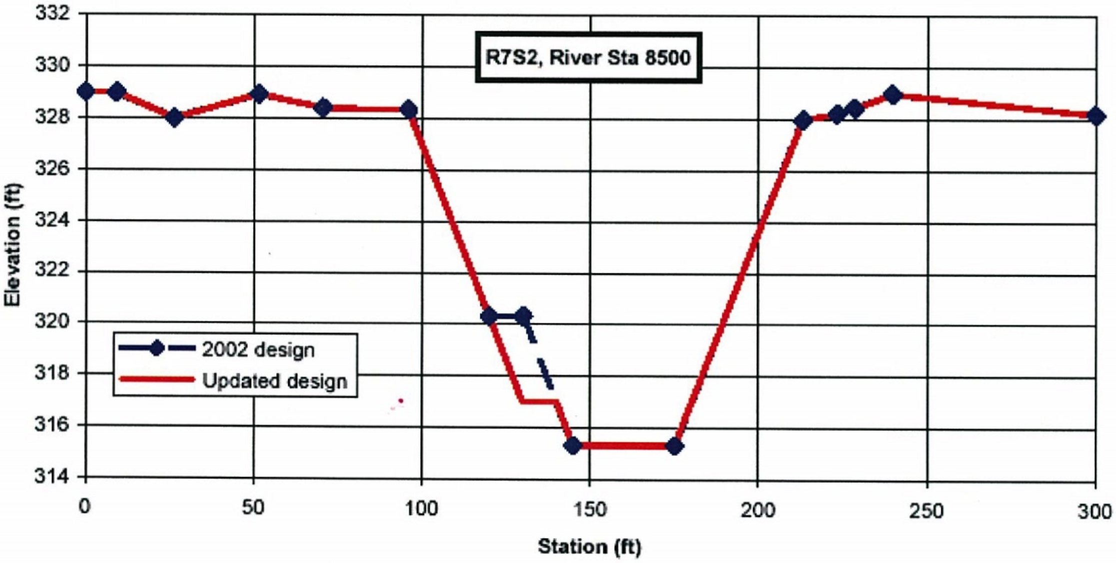


Figure 5-63. Example of Updated Channel Design Compared to 2002 Design for R7S2

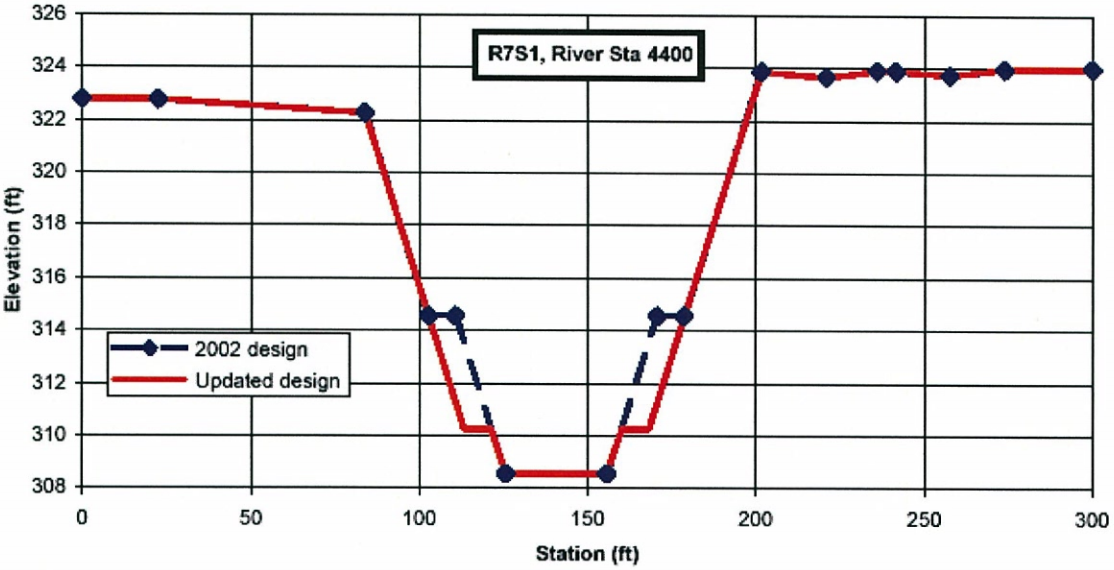


Figure 5-64. Example of Updated Channel Design Compared to 2002 Design for R7S1

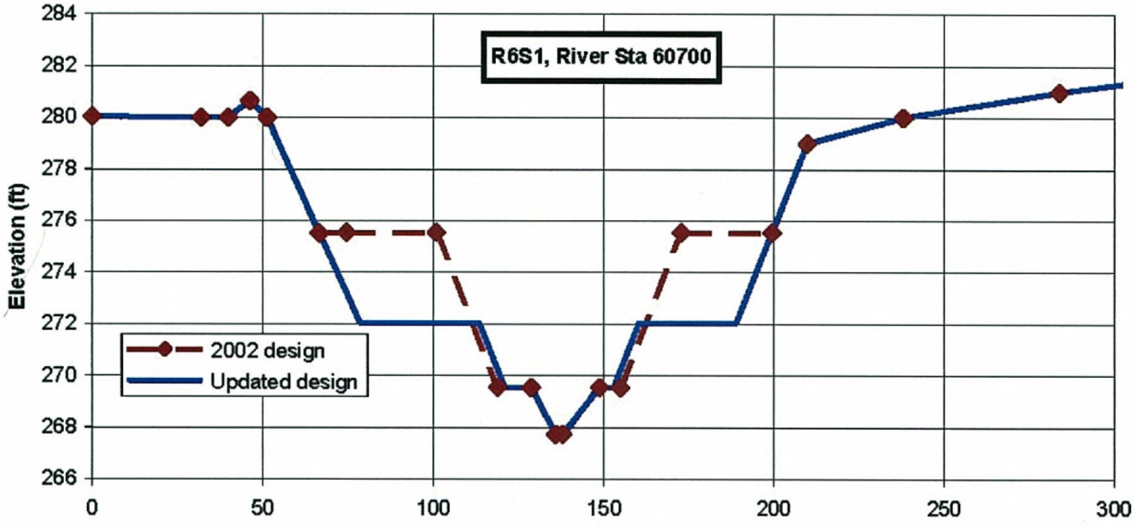


Figure 5-65. Example of Updated Channel Design Compared to 2002 Design for R6S1

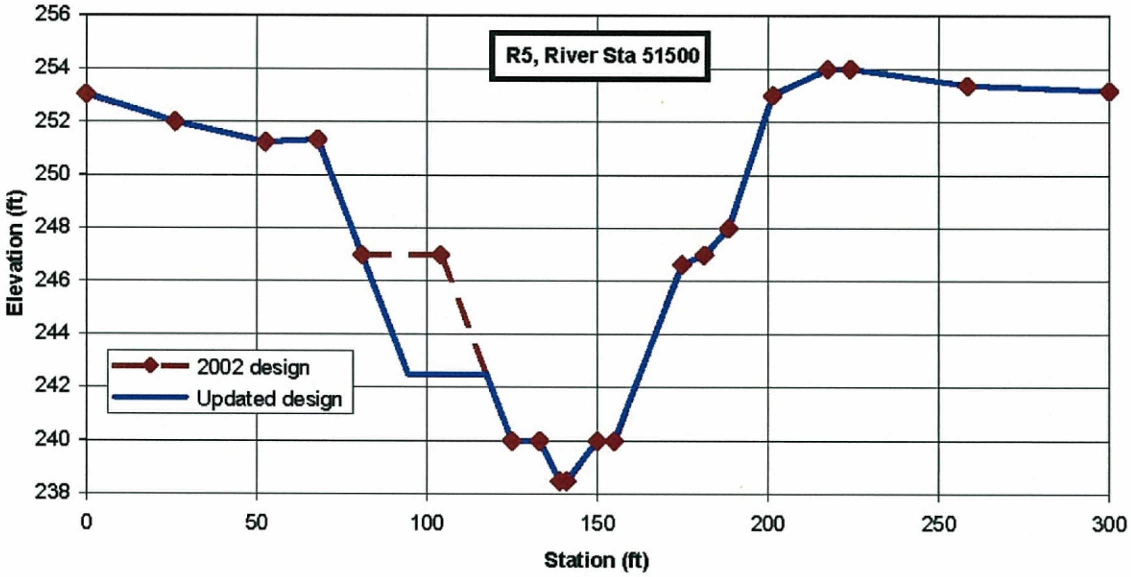
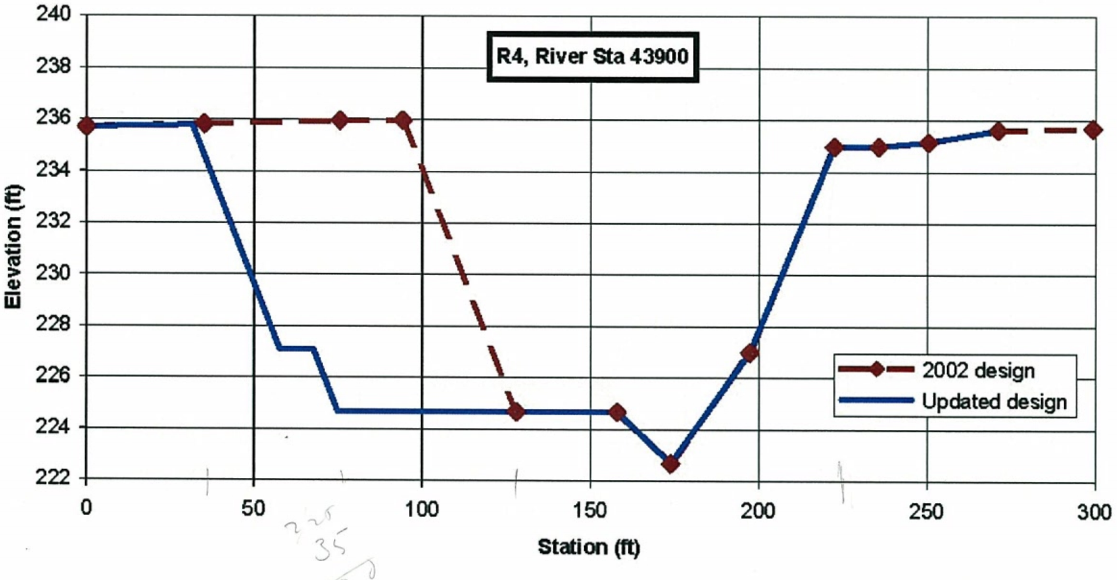


Figure 5-66. Example of Updated Channel Design Compared to 2002 Design for R5



225
35
170

Figure 5-67. Example of Updated Channel Design Compared to 2002 Design for R4

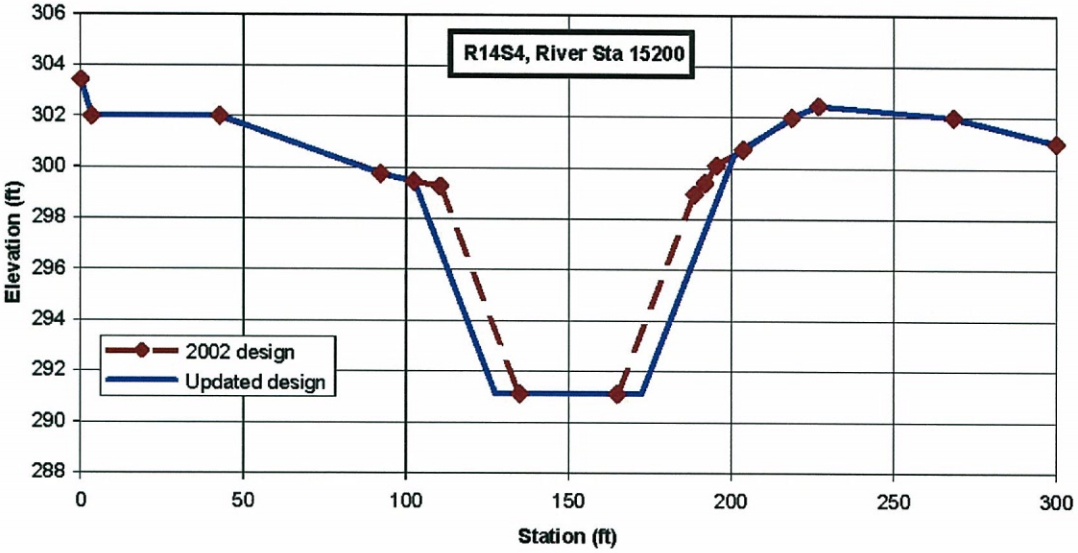


Figure 5-68. Example of Updated Channel Design Compared to 2002 Design for R14S4

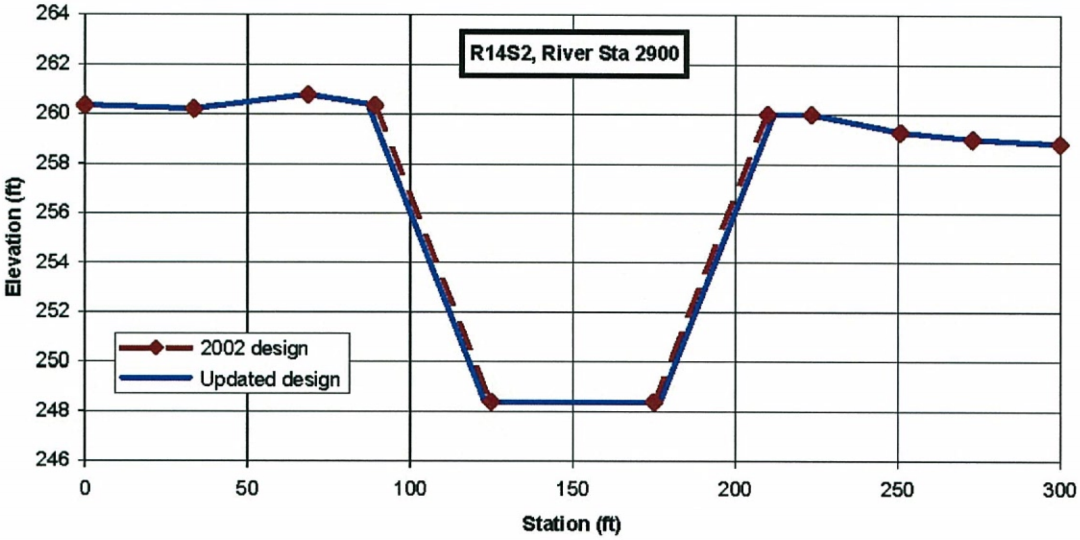


Figure 5-69. Example of Updated Channel Design Compared to 2002 Design for R14S2

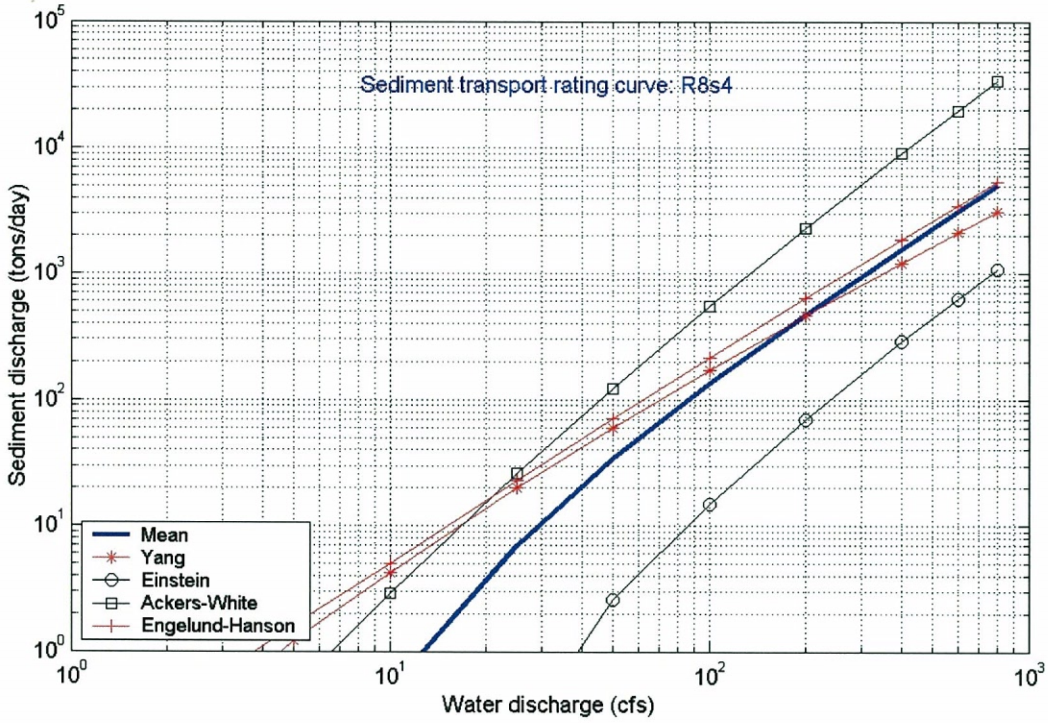


Figure 5-70. Updated Sediment Transport Rating Curve for Sub-Reach R8S4

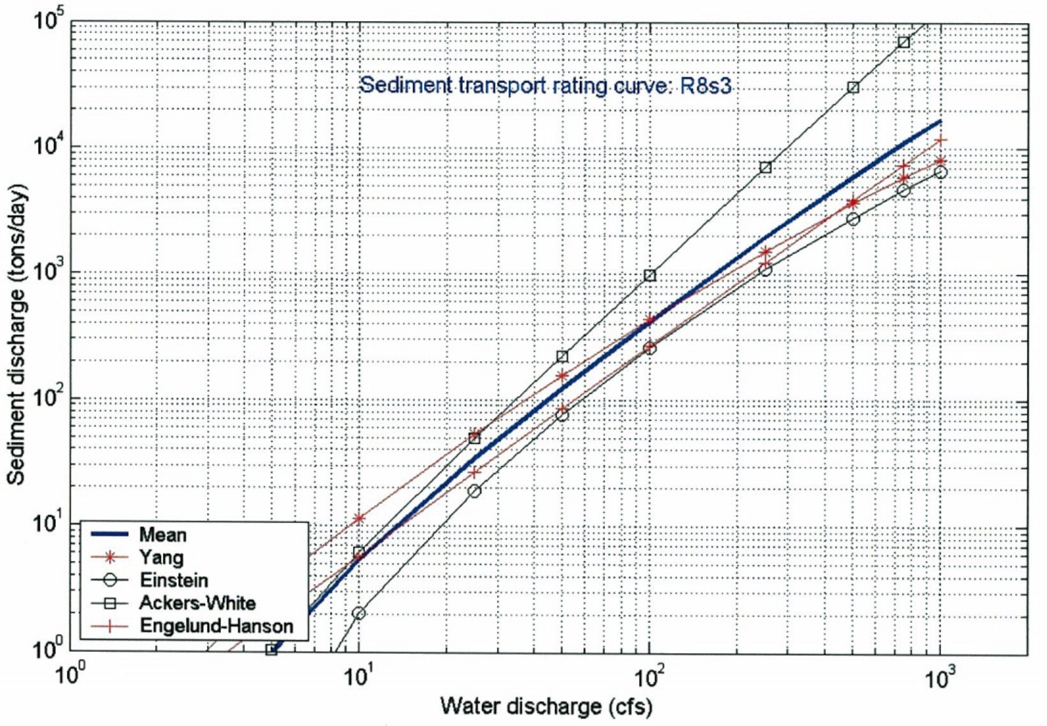


Figure 5-71. Updated Sediment Transport Rating Curve for Sub-Reach R8S3

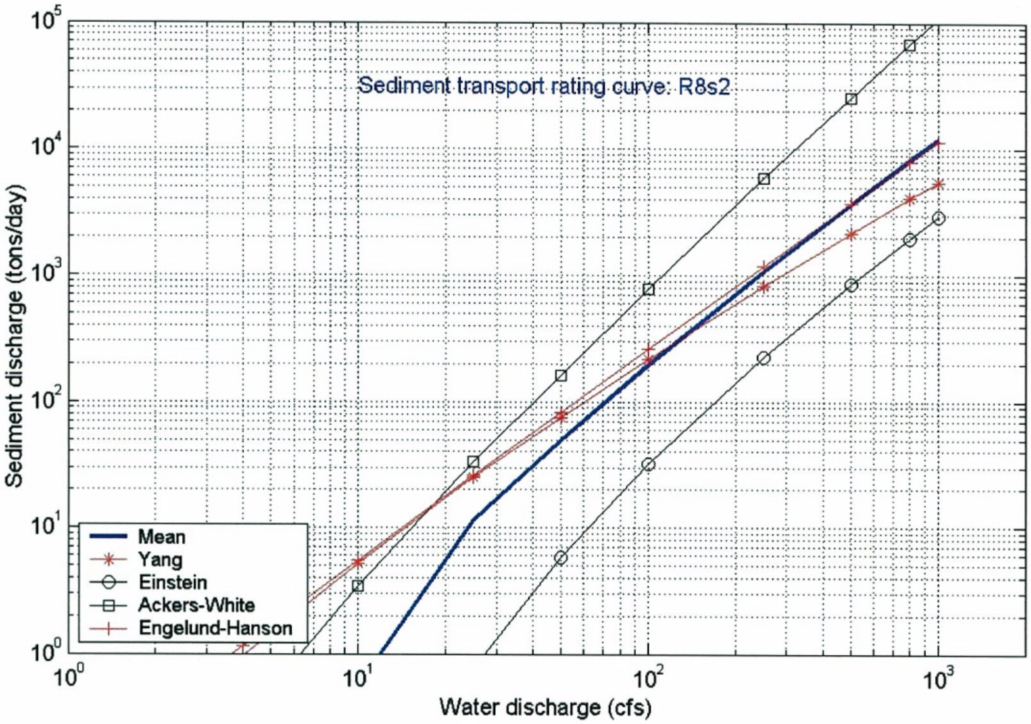


Figure 5-72. Updated Sediment Transport Rating Curve for Sub-Reach R8S4

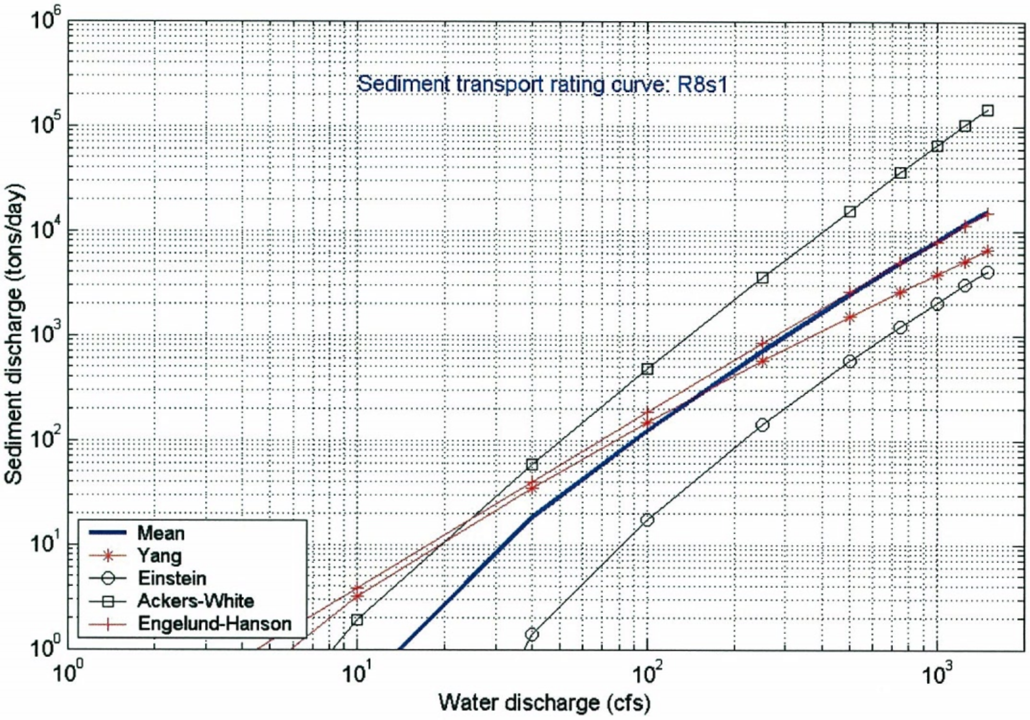


Figure 5-73. Updated Sediment Transport Rating Curve for Sub-Reach R8S1

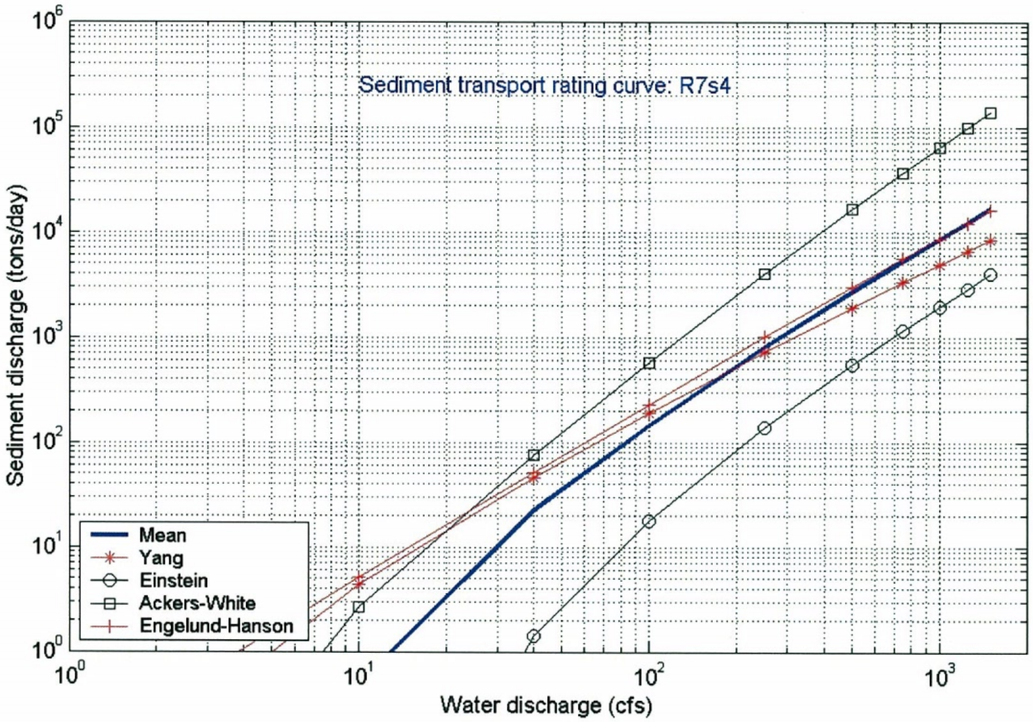


Figure 5-74. Updated Sediment Transport Rating Curve for Sub-Reach R7S4

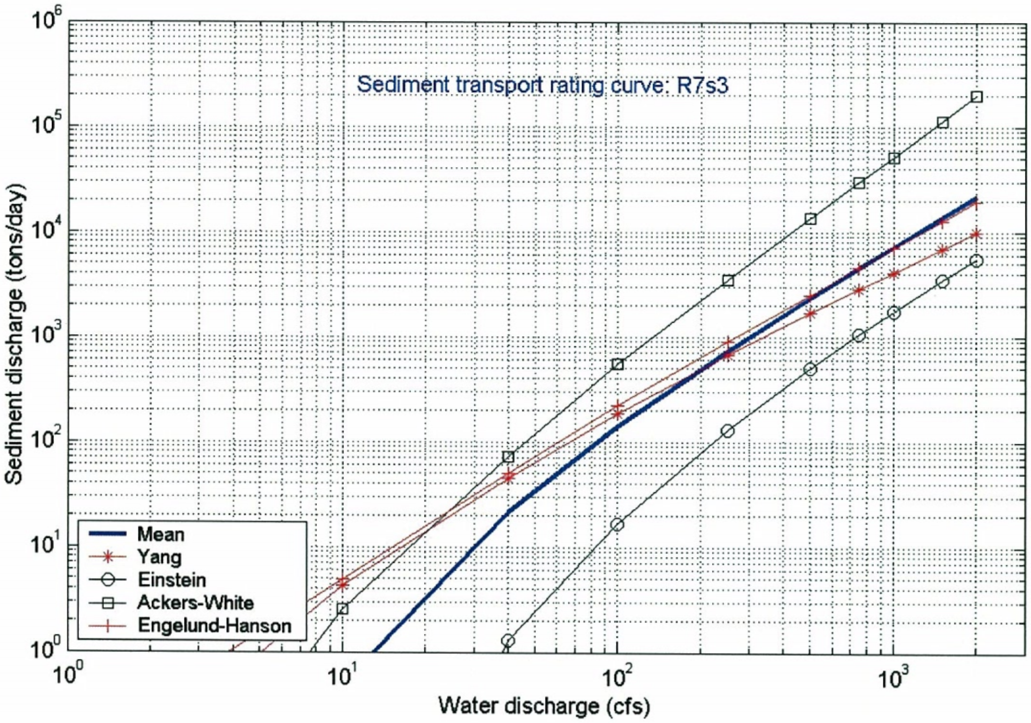


Figure 5-75. Updated Sediment Transport Rating Curve for Sub-Reach R7S3

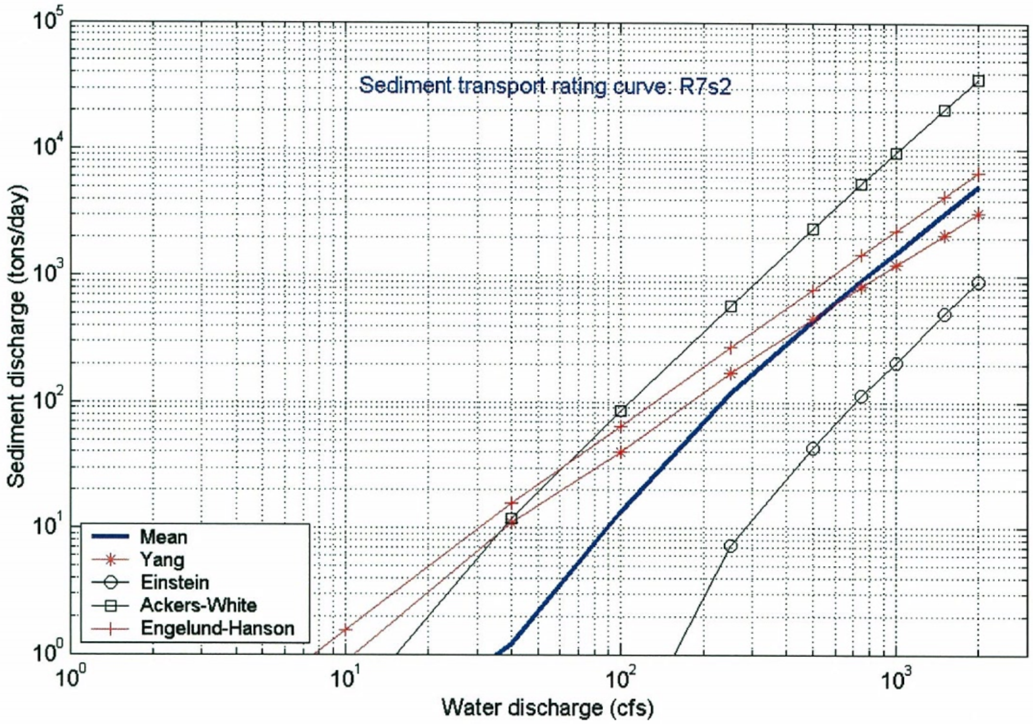


Figure 5-76. Updated Sediment Transport Rating Curve for Sub-Reach R7S2

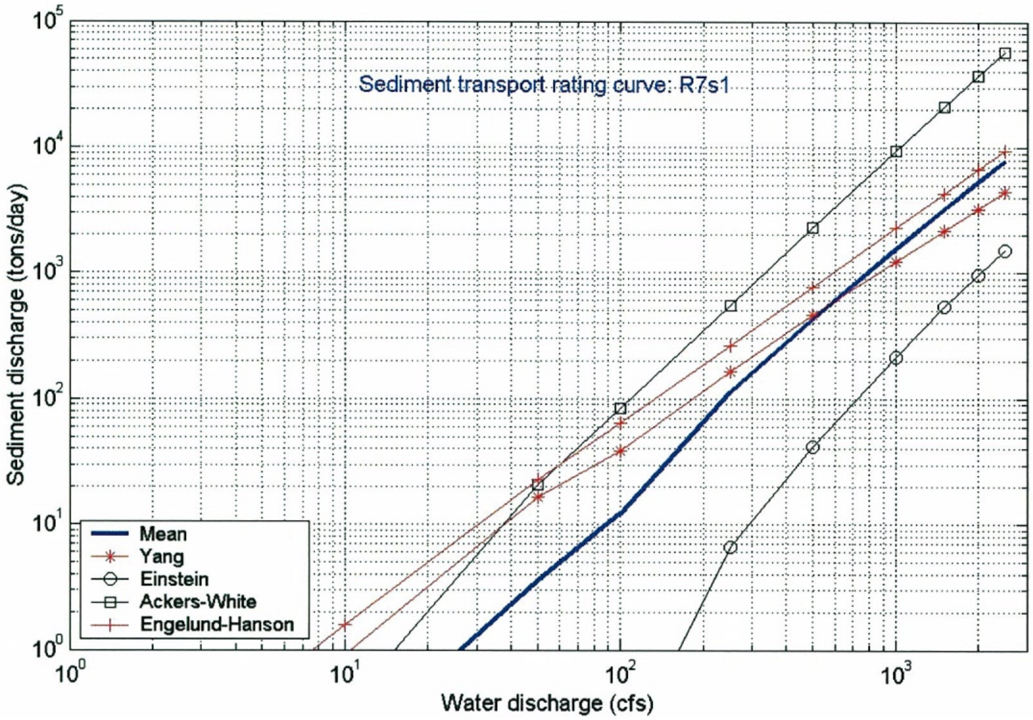


Figure 5-77. Updated Sediment Transport Rating Curve for Sub-Reach R7S1

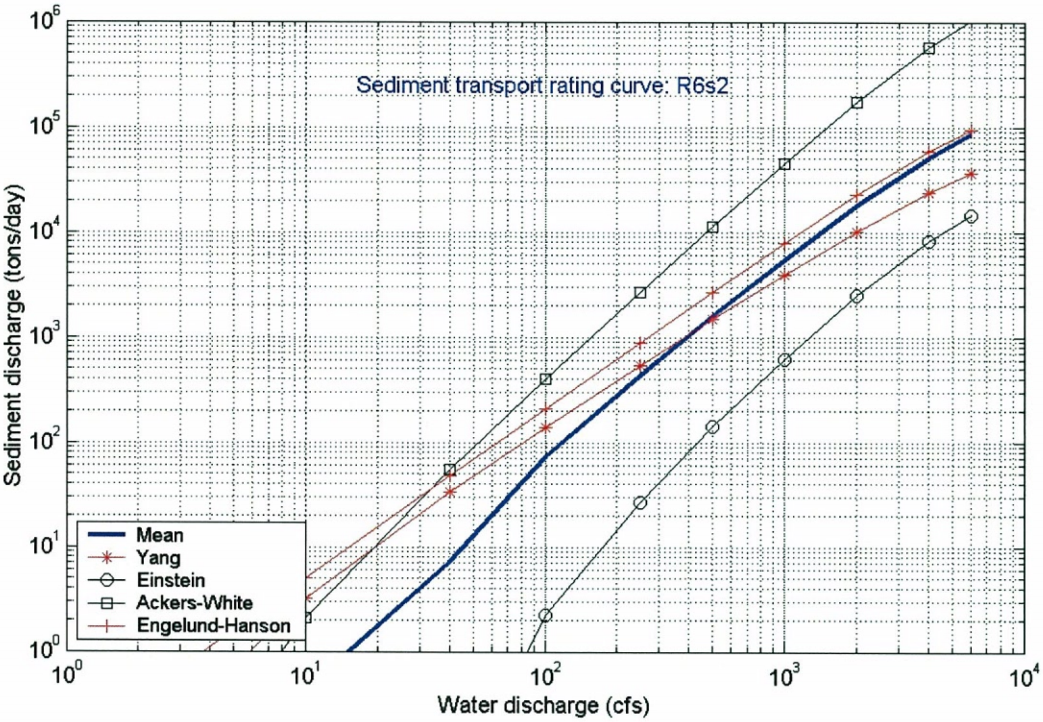


Figure 5-78. Updated Sediment Transport Rating Curve for Sub-Reach R6S2

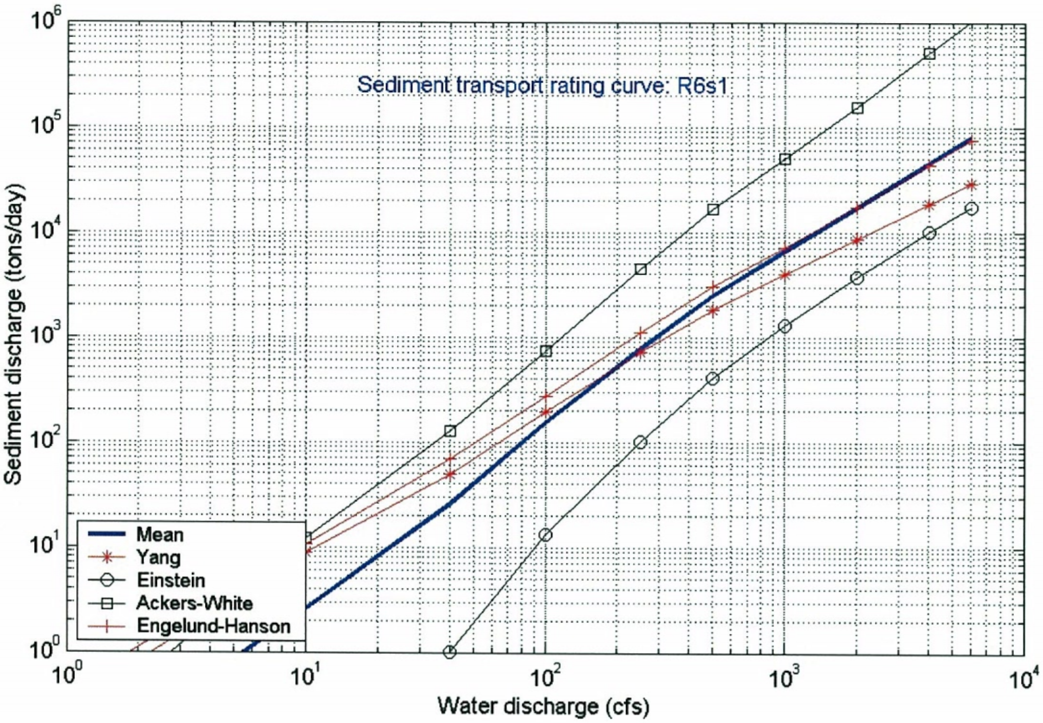


Figure 5-79. Updated Sediment Transport Rating Curve for Sub-Reach R6S1

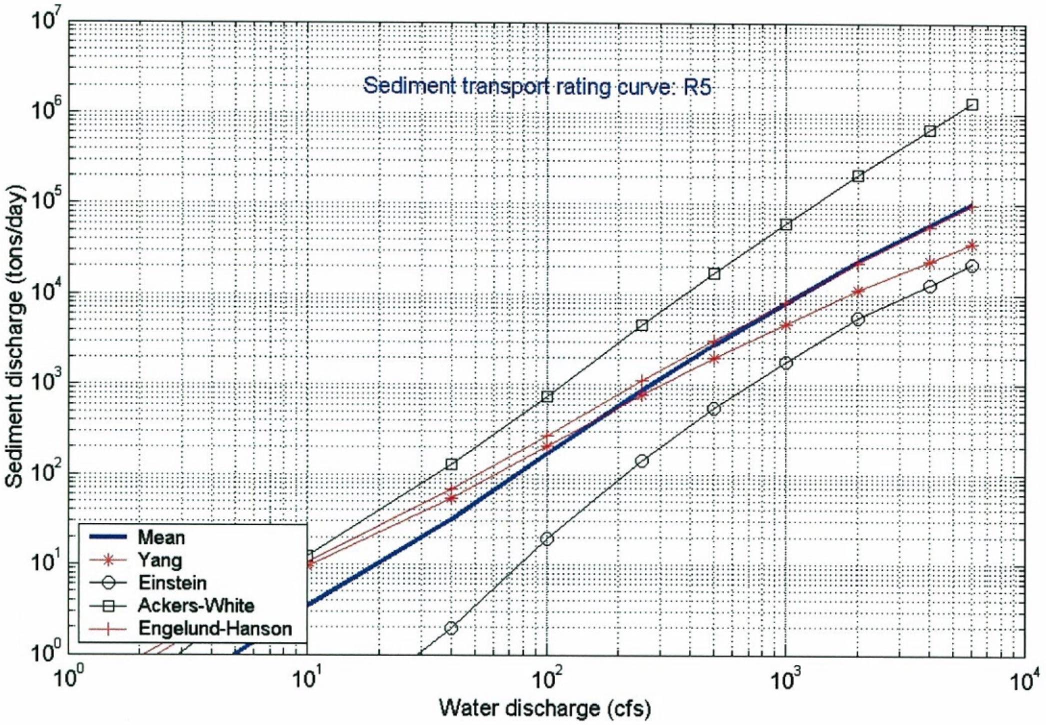


Figure 5-80. Updated Sediment Transport Rating Curve for Sub-Reach R5

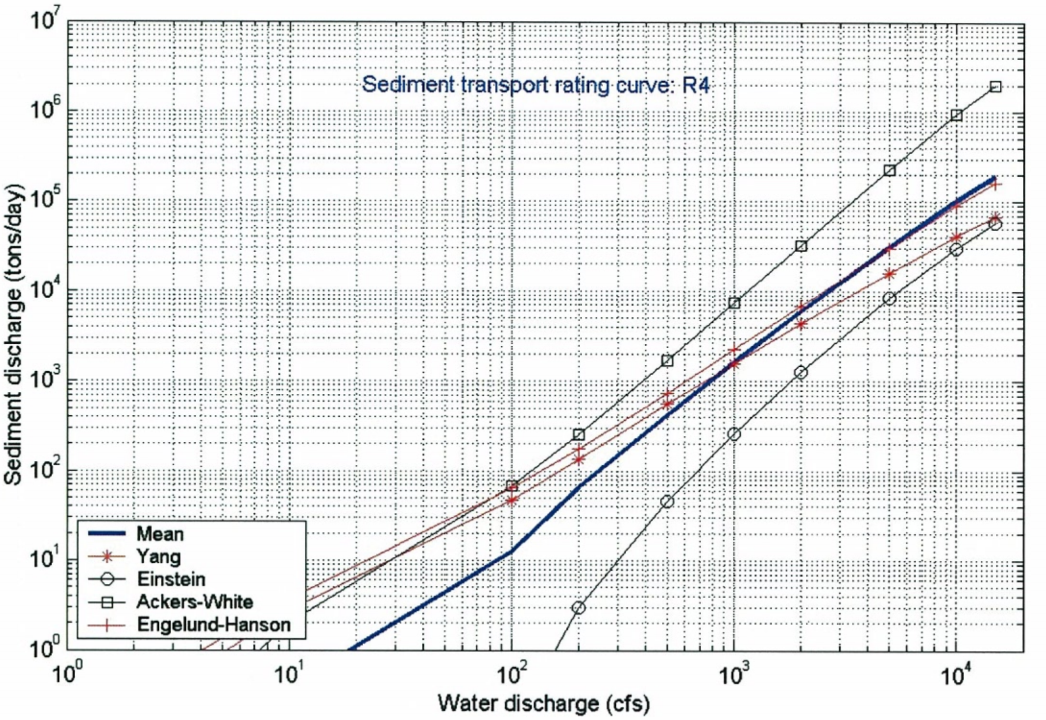


Figure 5-81. Updated Sediment Transport Rating Curve for Sub-Reach R4

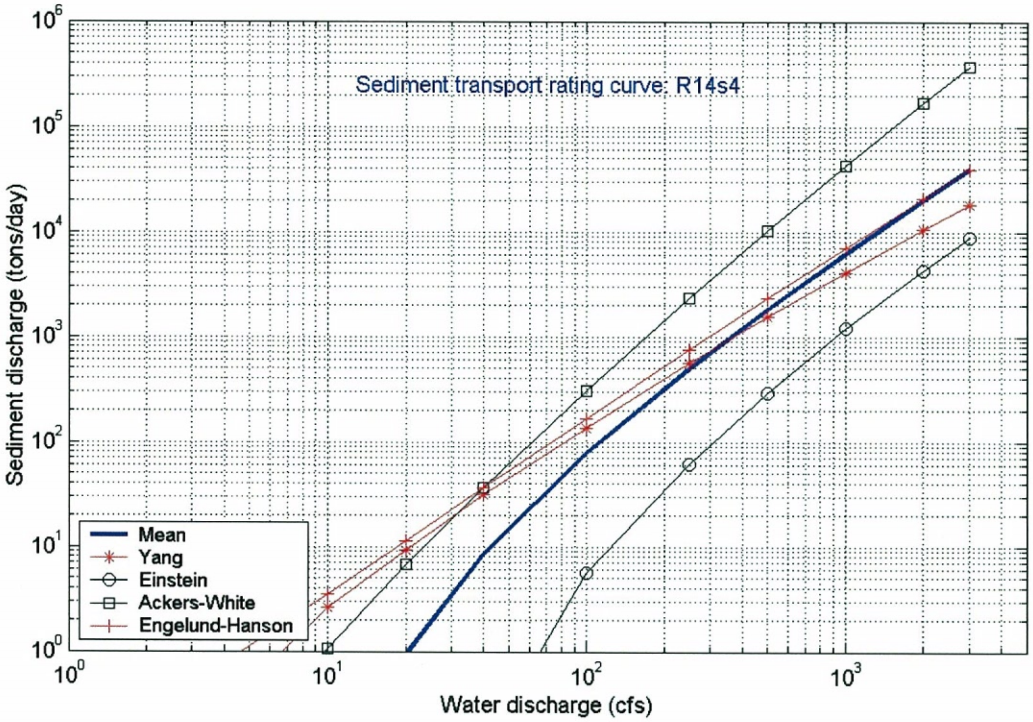


Figure 5-82. Updated Sediment Transport Rating Curve for Sub-Reach R14S4

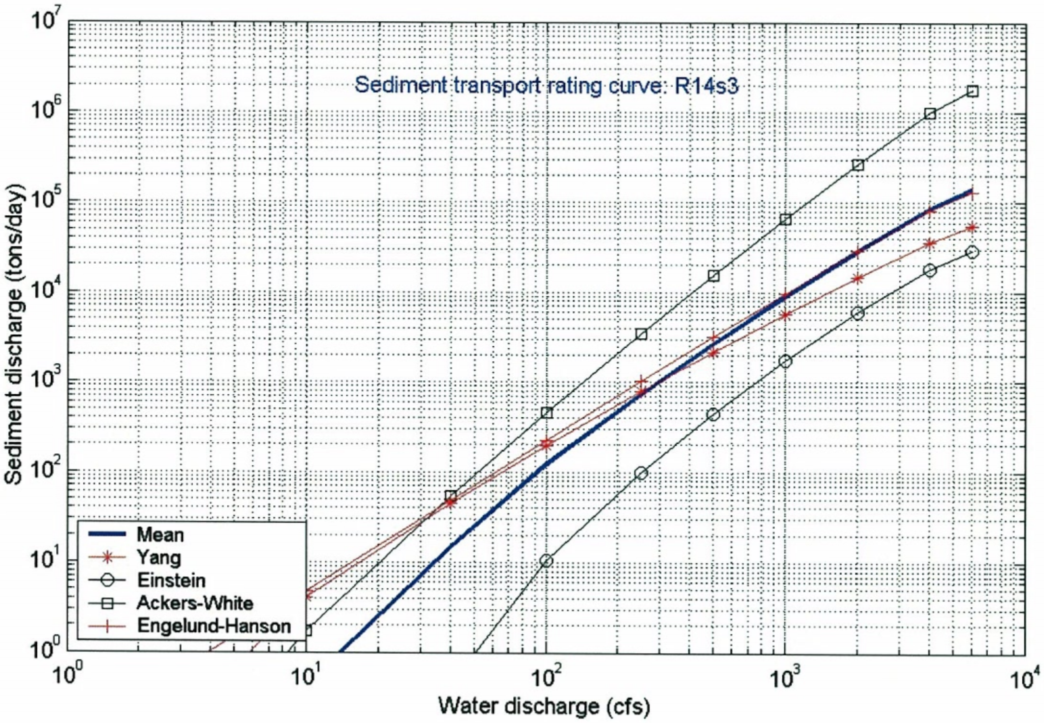


Figure 5-83. Updated Sediment Transport Rating Curve for Sub-Reach R14S3

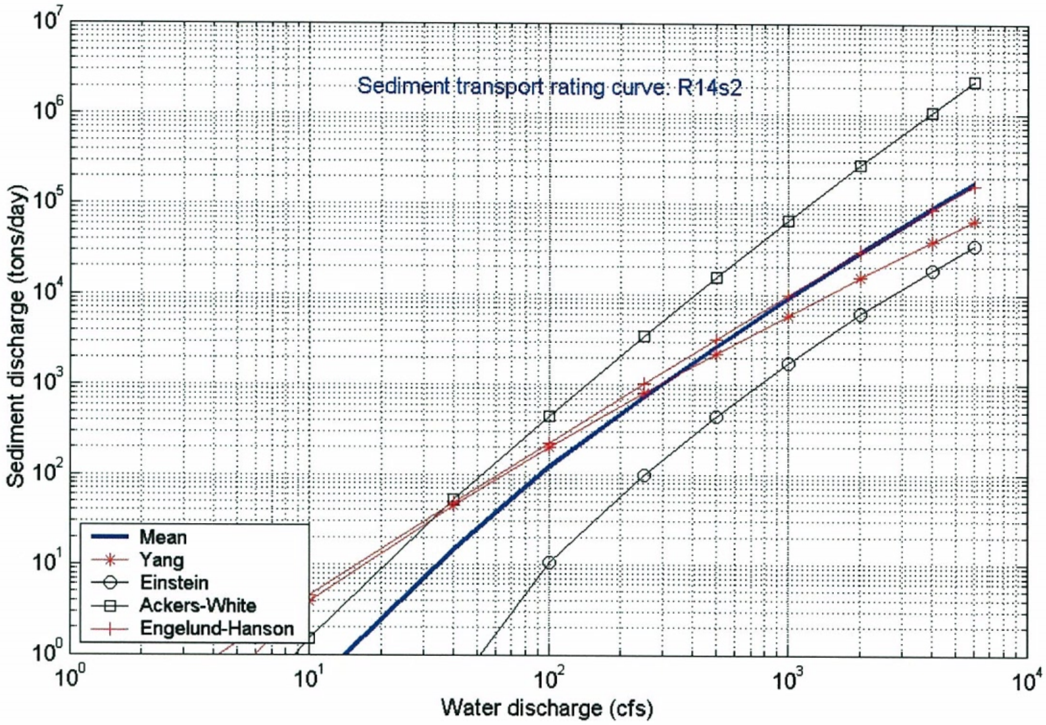


Figure 5-84. Updated Sediment Transport Rating Curve for Sub-Reach R14S2

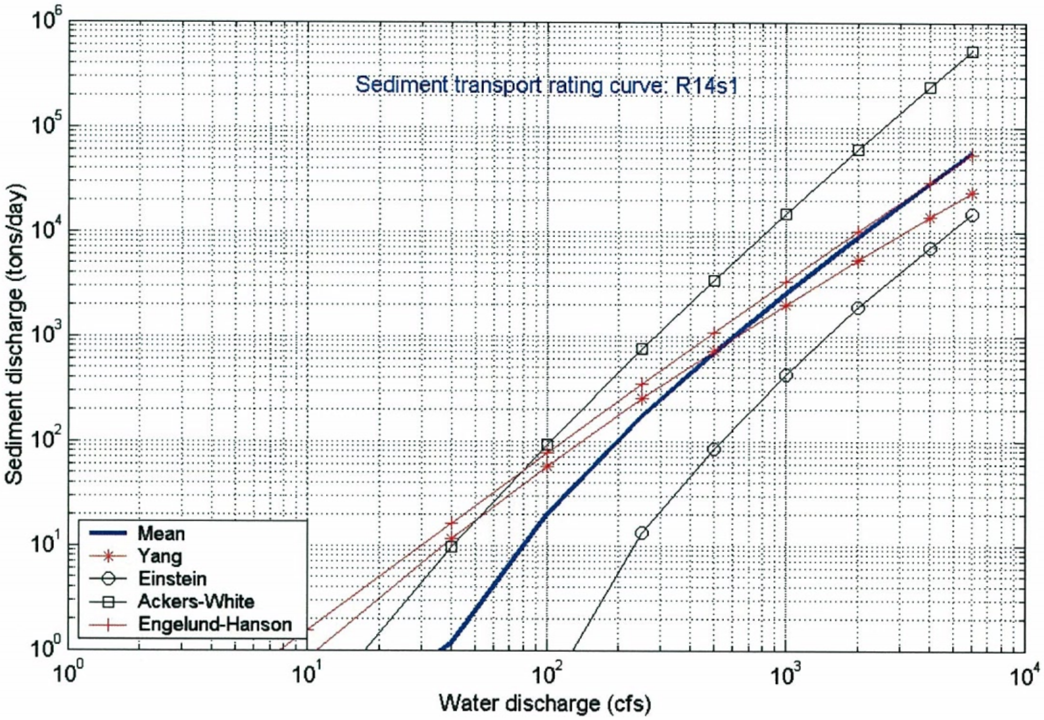


Figure 5-85. Updated Sediment Transport Rating Curve for Sub-Reach R14S1

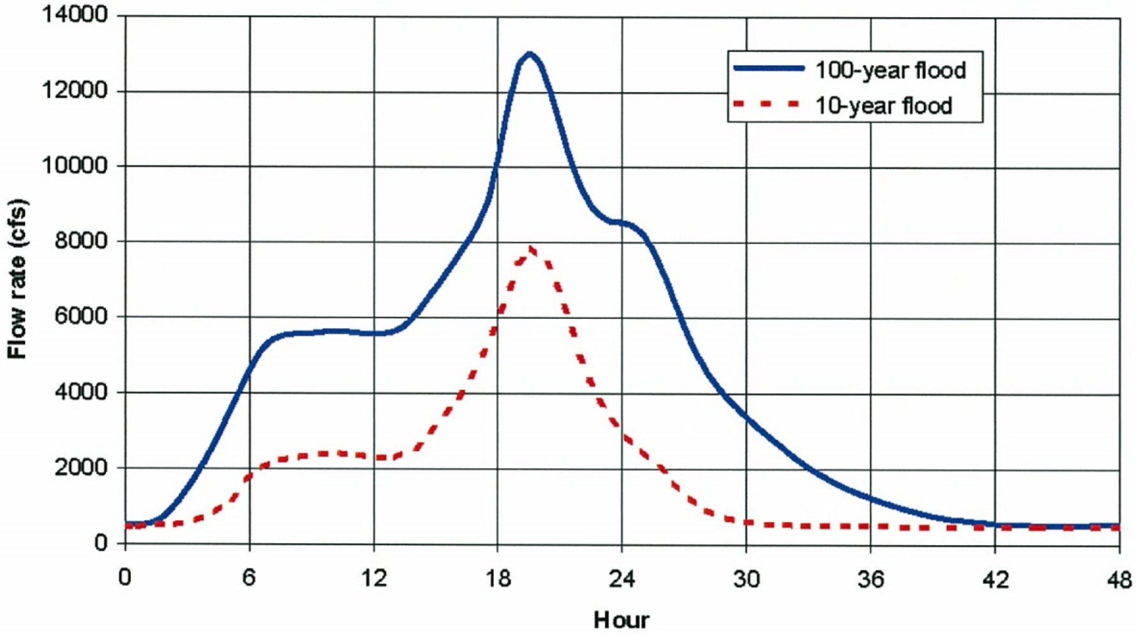


Figure 5-86. Estimated 10-Year and 100-Year Flood Hydrographs at Buena Vista Ave

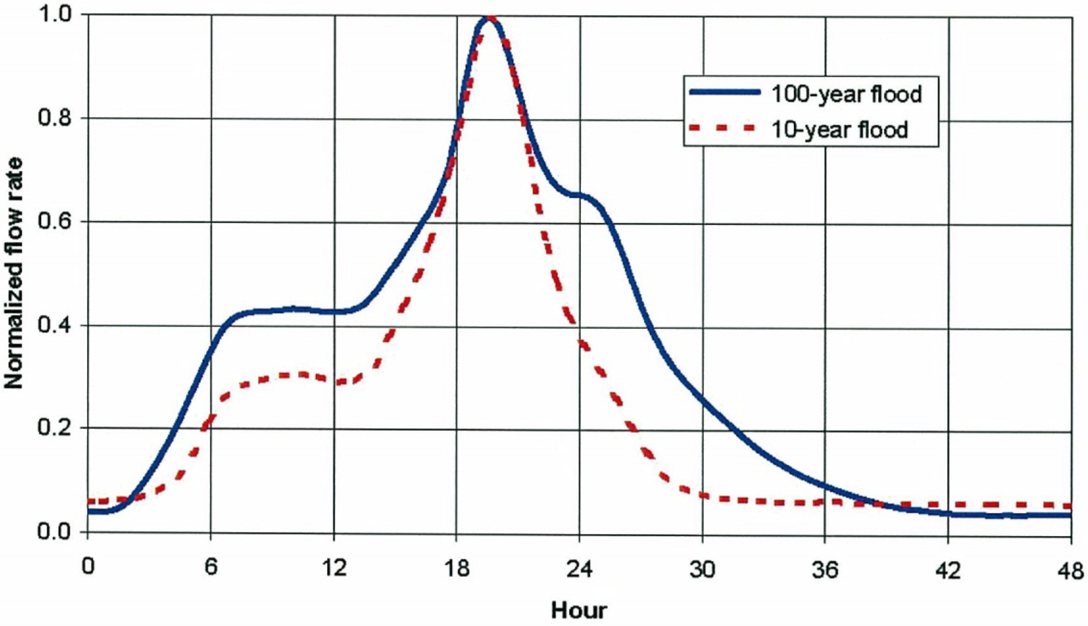


Figure 5-87. Representative Normalized 10-Year and 100-Year Flood Hydrographs

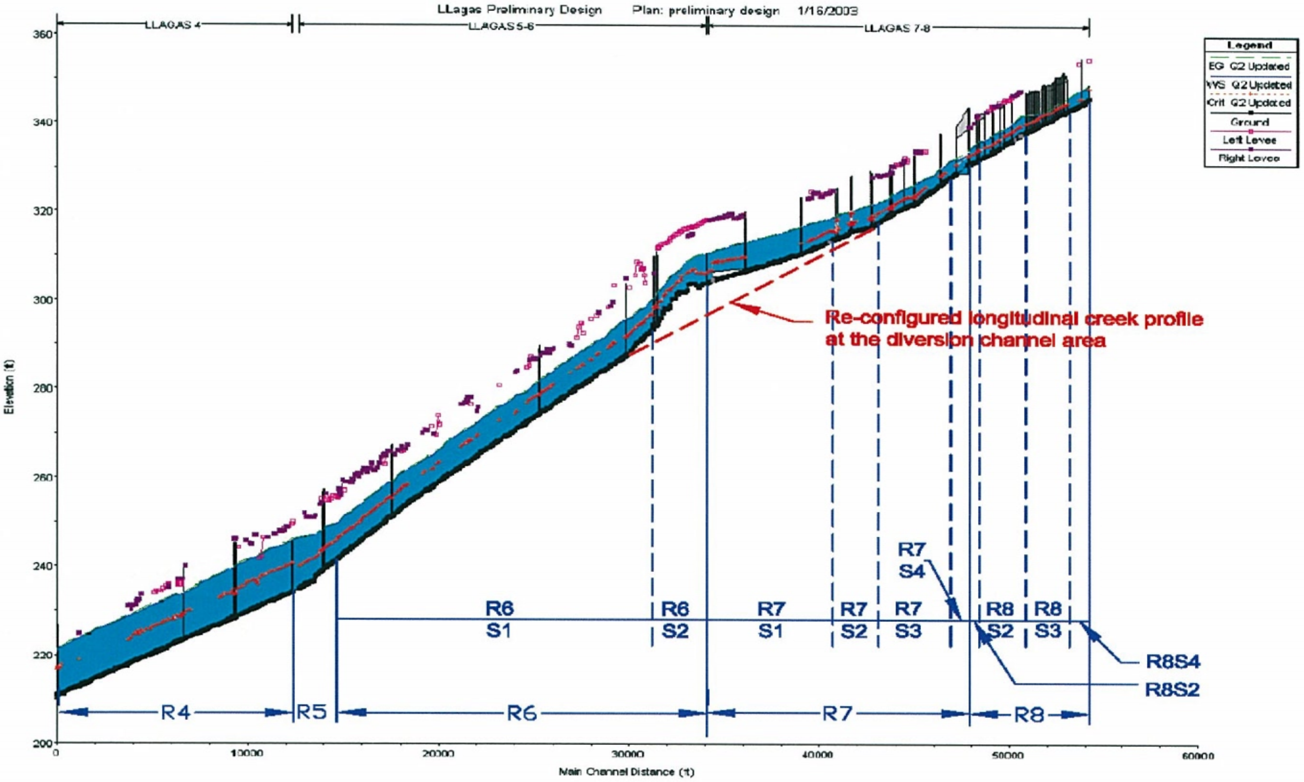


Figure 5-88. As-Design and Re-configured Longitudinal Creek Profile

6.0 FLOODPLAIN MAPPING

The Upper Llagas Creek Flood Control Project consists of channel improvements in Llagas Creek and the construction of a diversion channel as previously discussed in Section 1.0 of this report. The project reach schematic is shown in Figure 3-4. The channel geometry was modified in several reaches to improve the low-flow hydraulic conditions and to consequently lessen the requirement of maintenance operation in the creek channel. Inundation stages and floodplain boundaries under the modified channel conditions were determined for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year annual peak flows. The HEC-RAS model of Llagas Creek that was originally developed by the U.S. Army Corps of Engineers (USACE-SFD, 2002) was used for this floodplain mapping task. Modification to the original model was made to incorporate the improved low-flow channels, define the overbank floodplain sections, and simulate the project hydrology conditions developed by USACE (USACE-SFD, 2006). The floodplain boundaries were extracted from the HEC-RAS model using GeoRAS and were mapped in the ArcMap software.

The following sections briefly describe key features of the modified HEC-RAS model, discuss the model applicability for simulating the rare flow events, and, as an example, graphically present the inundation boundaries for the project conditions under the 100-year flow event. Other mapped flood boundaries under different flood conditions are electronically stored for use to evaluate the extent of the footprints for all simulated flood events.

6.1 Modification of HEC-RAS model

The HEC-RAS model of the Upper Llagas Creek system under the project conditions was originally developed in 2002 (USACE-SFD, 2002). The model includes (see Figure 3-4):

- West Little Llagas Creek below Llagas Road (Reach 8 and 7), including the diversion channel connecting West Little Llagas Creek and Llagas Creek near Lake Silveria;
- Llagas Creek between Lake Silveria and the confluence with East Little Llagas Creek (Reaches 6 and 5);
- Llagas Creek between the confluence with East Little Llagas Creek and Buena Vista Avenue (Reach 4); and
- East Little Llagas Creek below Middle Road (Reach 14).

The channel geometry of the original HEC-RAS model under the project conditions incorporates channel features that extend only a few hundred feet beyond the channel banks (the cross sections do not include the existing floodplain areas). In addition, several cross-sections in the original with-project HEC-RAS model developed by USACE were not geo-referenced. These cross-sections that do not have associated spatial coordinates are listed in **Table 6-1**. Spatial inconsistency between the flow centerline and cross-section bank stations is also apparent in some of the model reaches, as shown in **Figure 6-1**.

Table 6-1. Cross-Sections in With-Project HEC-RAS Model Lacking Spatial Coordinates

Model reach	7-8	7-8	7-8	5-6	4	4
Station	148+75.73	148+35.34	140+82.82	581+00	377+77	377+32

The overbank portion of the cross-sections in the project model was extended into the floodplains to properly incorporate floodplain conveyance. Floodplain areas were added to the original with-project cross-sections using the corresponding cross-sections from the HEC-RAS model for the existing conditions in which the HEC-RAS model did incorporate floodplain geometry. Spatial coordinates of the extended project cross-sections were taken from the corresponding existing cross-sections. Since the without-project HEC-RAS model utilized fewer cross sections than the original with-project model, the with-project cross-sections, which could not be extended from the existing condition data, were removed from the model geometry. The cross-sections without any geo-referencing as well as the skewed cross-sections causing the intersection problems were also removed from the project geometry in the HEC-RAS model.

The 2002 HEC-RAS model was modified to incorporate the new low-flow channel designs in various reaches of Llagas Creek, as presented in Section 5.0. Low flow cross-sections were added in some reaches or modified in other reaches to accommodate specific hydraulic needs as well as to comply with the site conditions. The low-flow channel modification, including the bottom width, height, bank slope and bench width, extends to Sub-reaches R7S3, R7S2, R7S1, R6S1, R14S4, R14S2, and the entire Reaches 5 and 4 (see Table 5-2). The examples of the updated channel designs for various sub-reaches are shown in Figures 5-62 through 5-69. The floodplain areas were also expanded beyond the original model's footprint to better simulate the bank overflow conditions, if occurred. Peak flows for the with-project conditions in the HEC-RAS model were set in accordance with the derived flow rates that were presented in Table 5 of the USACE report (USACE-SFD, 2006).

Model simulations indicate that at the extremely high flows (e.g. return periods equal to or exceeding 100 years), floodwaters would overflow the creek channel into the floodplain areas at various locations, particularly in the lower reach (i.e., Reach 4) below the confluence of Llagas Creek with East Little Llagas Creek. It is noted that the channel cross-sections in the original with-project HEC-RAS model are restricted to the main channel only and do not include the existing floodplains. As a consequence, the model underestimated the floodplain conveyance and resulted in overestimating the computed flood levels as well as underestimating the simulated inundation area. If there is no flood wall with an adequate height, then overestimating the computed flood levels would cause overestimate of the simulated inundation area. **Figure 6-2** illustrates the effect of the spatial extent of the modeled cross-sections on the predicted water levels for the 100-year peak flow. Incorporating floodplain conveyance by expanding the floodplain portion in the original HEC-RAS model for the project conditions, the simulated water surface elevations were reduced by about 0.2 to 0.4 feet in the middle reach (Reaches 5, 6 and

14) and by around 1.5 feet in the lower reach (Reach 4). Concurrently, the computed inundation areas were expanded by a hundred feet in the middle reach and by over a thousand feet in the lower reach of the model.

6.2 Floodplain mapping

Floodplain mapping in the study reach under the project conditions was simulated using two formulated HEC-RAS models:

- The "Project modified" (PM) model, which is based on the original 2002 USACE with-project model, incorporates the designed new low-flow channel, and uses the revised hydrology.
- The "Project modified extended" (PME) model, which is the above-described model with the inclusion of the floodplain areas that was not included in the 2002 USACE with-project model.

In the absence of a digital elevation model representing the ground topography under the project conditions, only the floodplain boundaries were extracted from the HEC-RAS models and plotted in ArcMap.

Figure 6-3 shows the 100-year inundation boundaries simulated using the PM model for the entire project area. **Figure 6-4, Figure 6-5** and **Figure 6-6** respectively show the 100-year inundation areas simulated for the upper, middle, and lower project reaches. It is obvious from these figures that the use of the PM model is inadequate for mapping high flows exceeding the capacity of the main channel. The simulated 100-year inundation area in Figures 6-3 through 6-6 is limited by the spatial extent of the modeled cross-sections rather than by the physical floodplain boundaries.

Figure 6-7 shows the 100-year inundation boundaries simulated using the PME model for the entire project area. The 100-year inundation areas that were simulated for the upper, middle, and lower project reaches are respectively shown in **Figure 6-8**, **Figure 6-9** and **Figure 6-10**. In these figures, the inundation area is not limited by the model boundaries, although the level of detail is lower than that produced by the PM model due to fewer cross-sections included in the PME model.

The original with-project HEC-RAS model does not include the floodplain geometry and, therefore, is inadequate for mapping the 100-, 250-, and 500-year flows that exceed the capacity of the main channels, particularly in the lower reach (Reach 4). In such a "restricted" model, flood levels are overestimated and the spatial extent of the inundation area is significantly underestimated. For a proper mapping of the 100-, 250-, and 500-year floodplains, the model cross-sections must include the entire floodplain area. The lack of a digital elevation model representing the with-project ground topography only allow for the mapping of the inundation areas, but not the depths, in GeoRAS. For a flow event that has a recurrence interval of less than 100-years, floodwaters are mostly contained within the main channel and the floodplain is confined to the channel cross sections only.

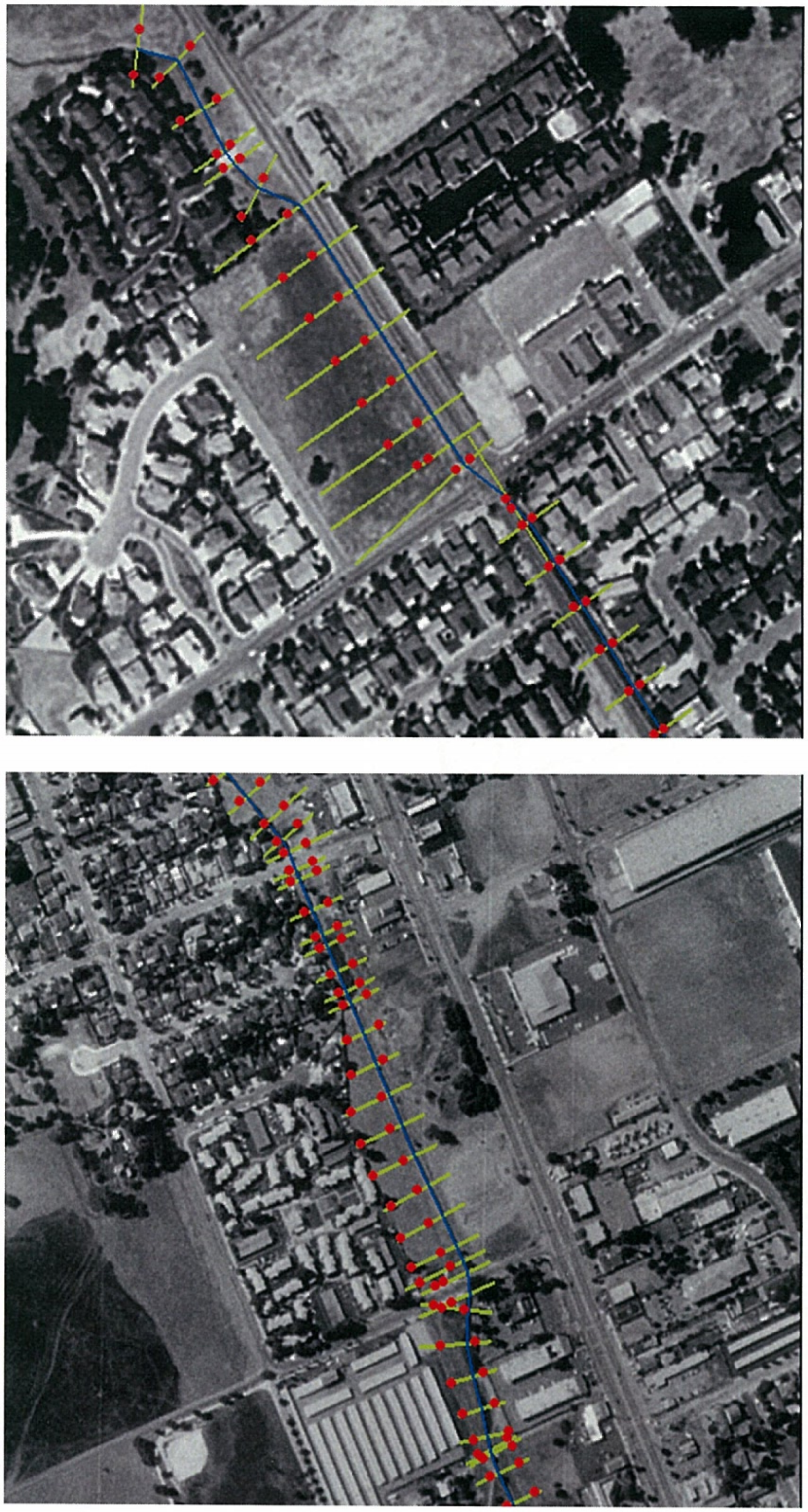


Figure 6-1. Examples of Intersecting Cross-Sections and Spatial Inconsistency between Flow Centerline (Line) and Bank Stations (Dots) in Model Reach 7-8

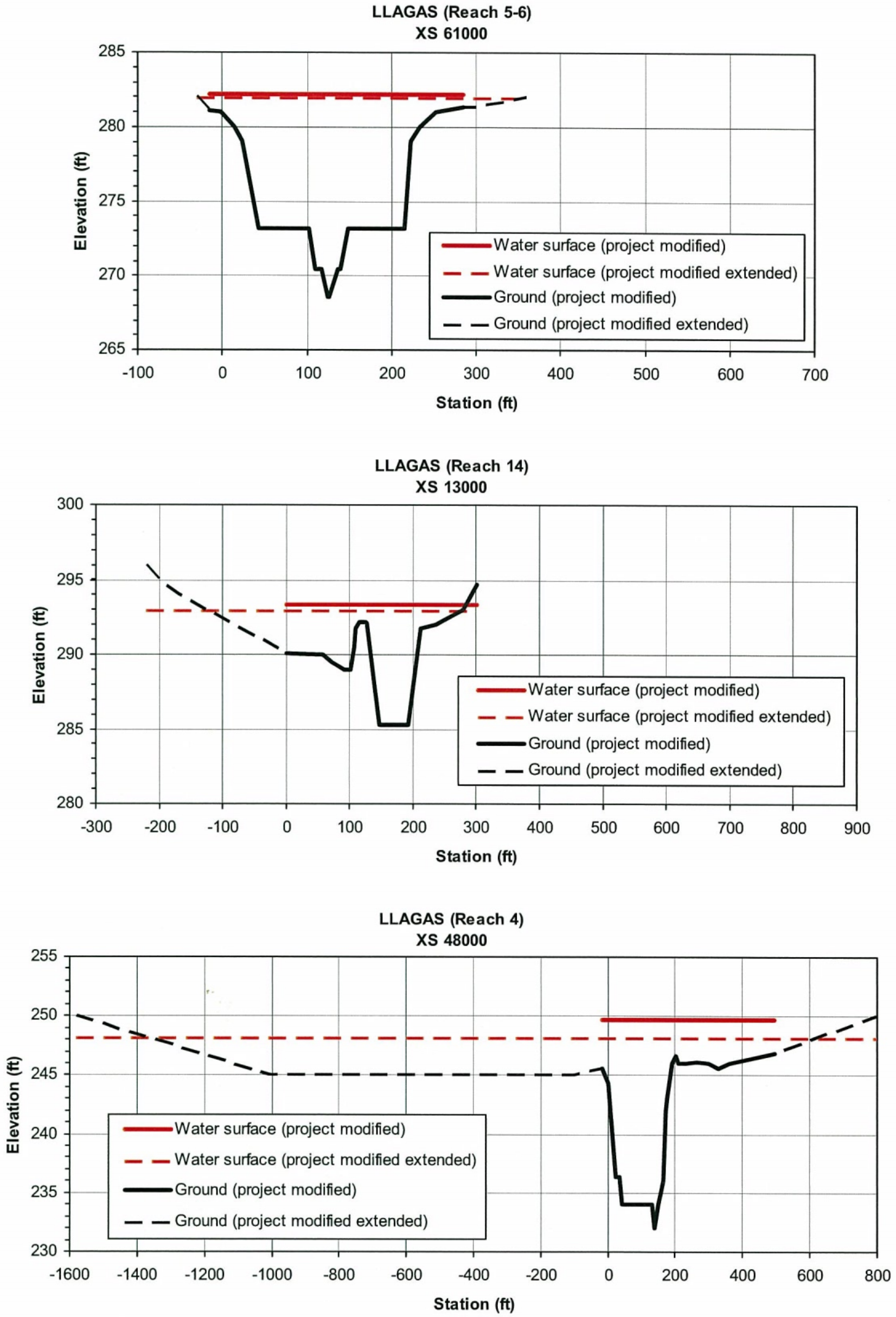


Figure 6-2. Comparison of 100-year Water Surface Elevations Simulated for Original and Extended With-Project Cross-Sections in HEC-RAS Model

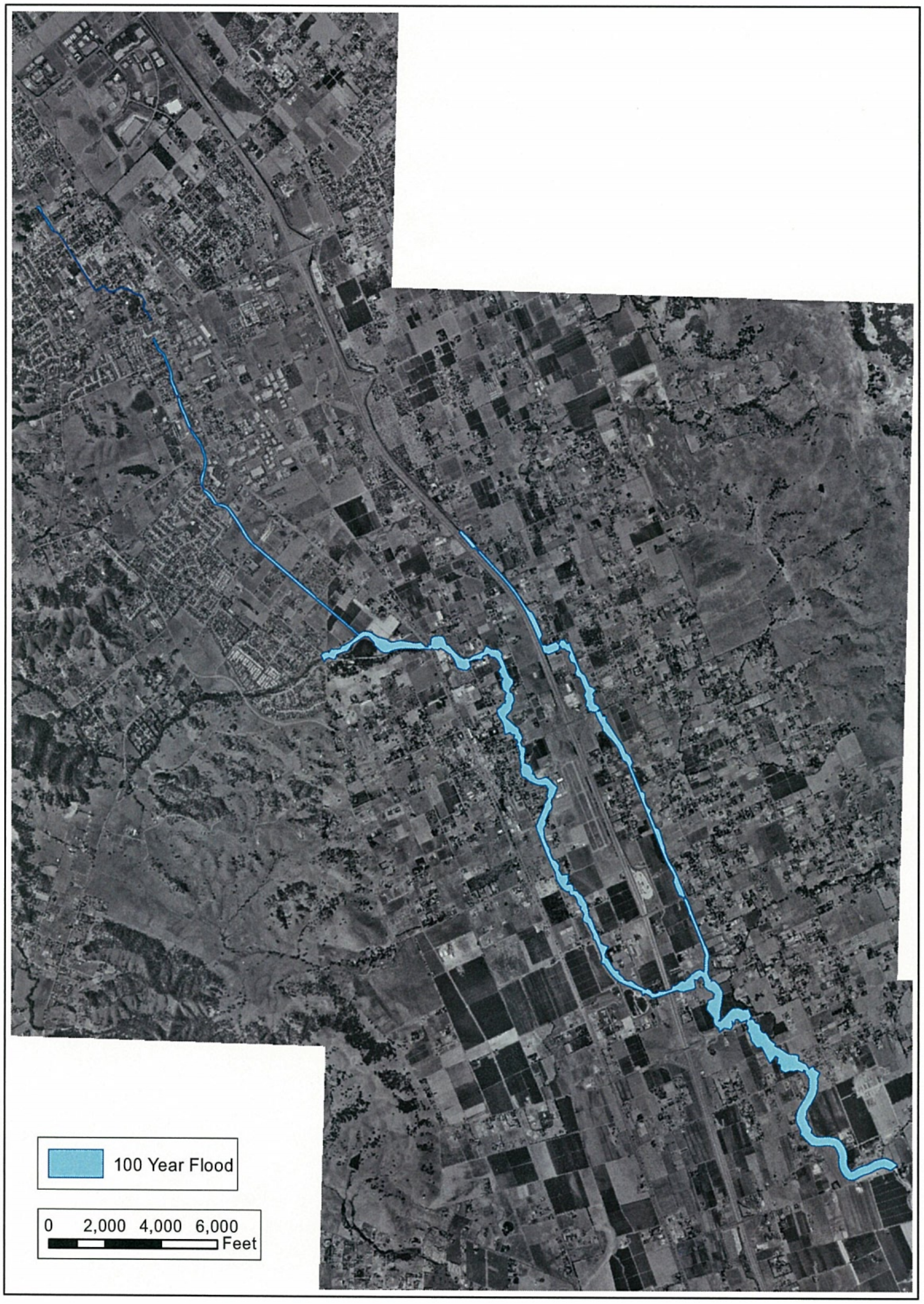


Figure 6-3. 100-year PM Inundation Boundaries in The Entire Reach

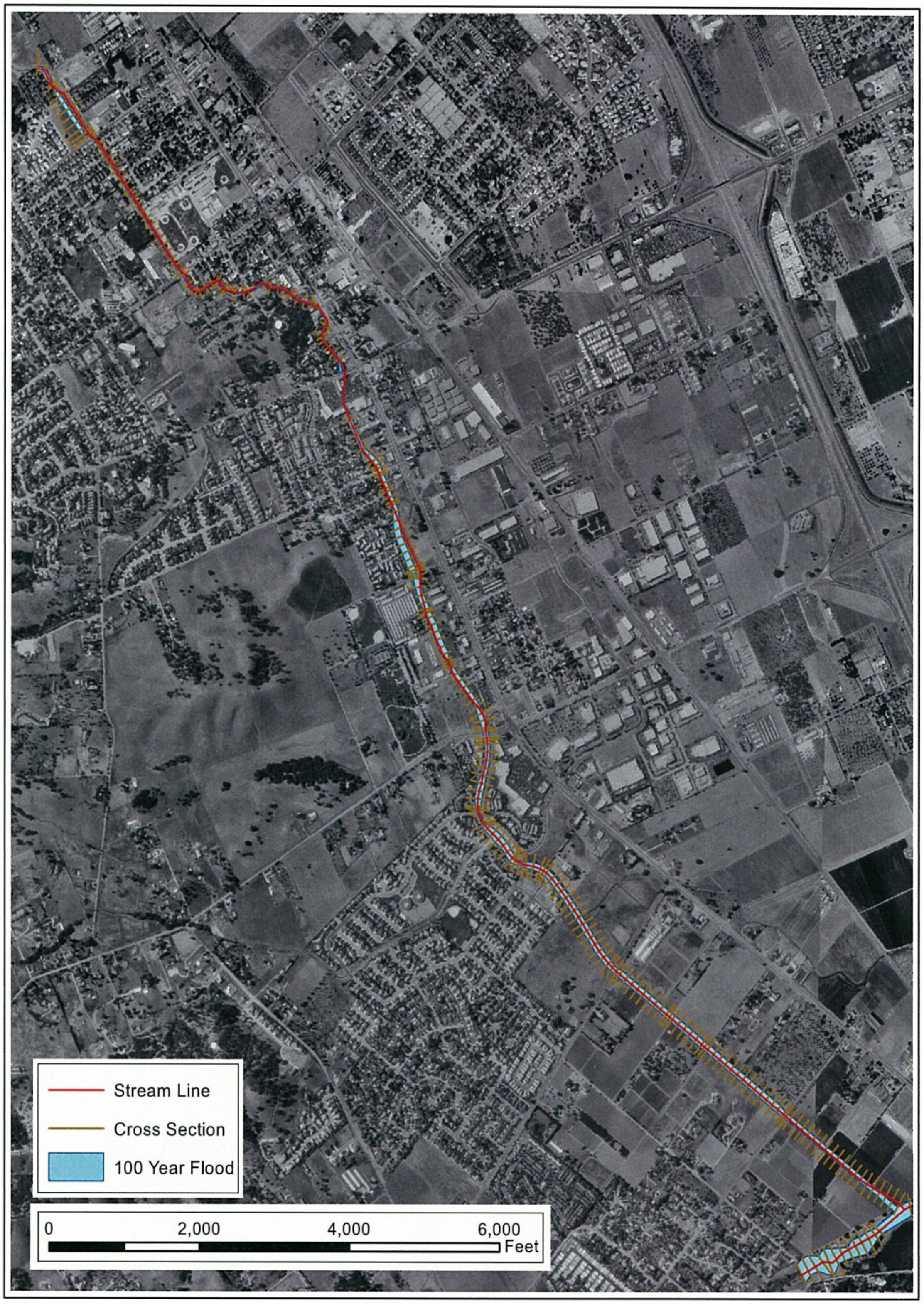


Figure 6-4. 100-year PM Inundation boundaries in the Upper Reach

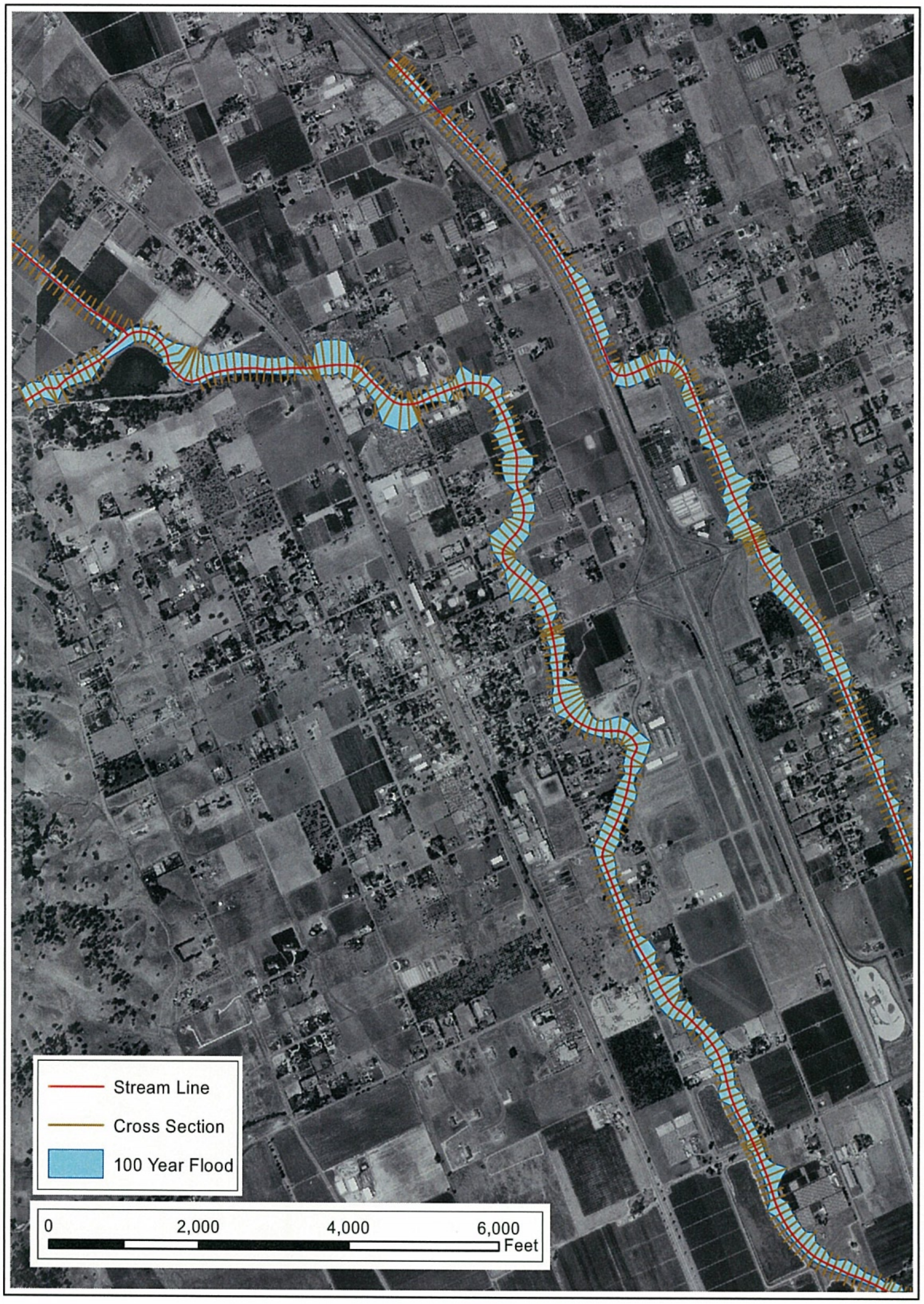


Figure 6-5. 100-year PM Inundation Boundaries in the Middle Reach

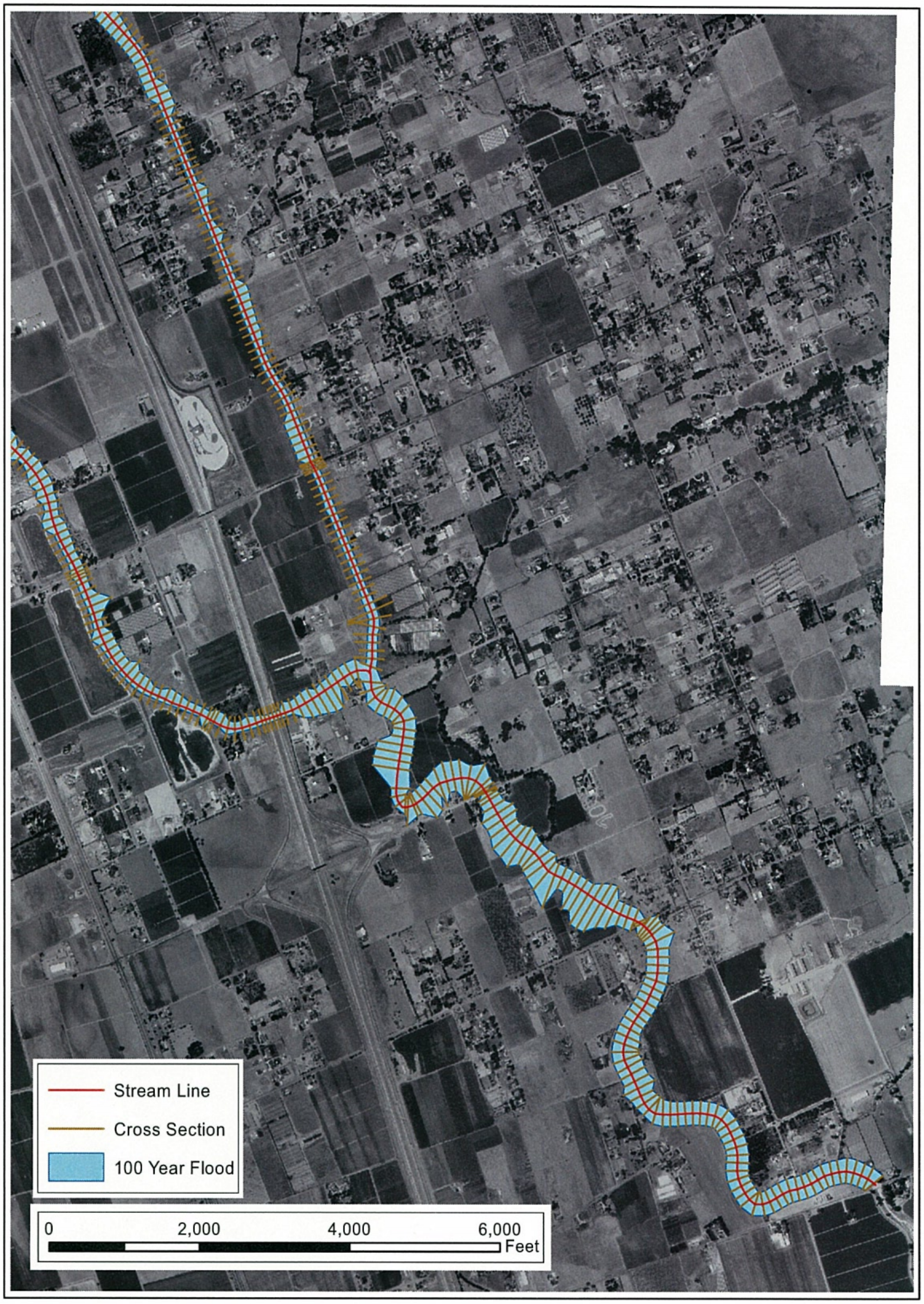


Figure 6-6. 100-year PM Inundation Boundaries in The Lower Reach

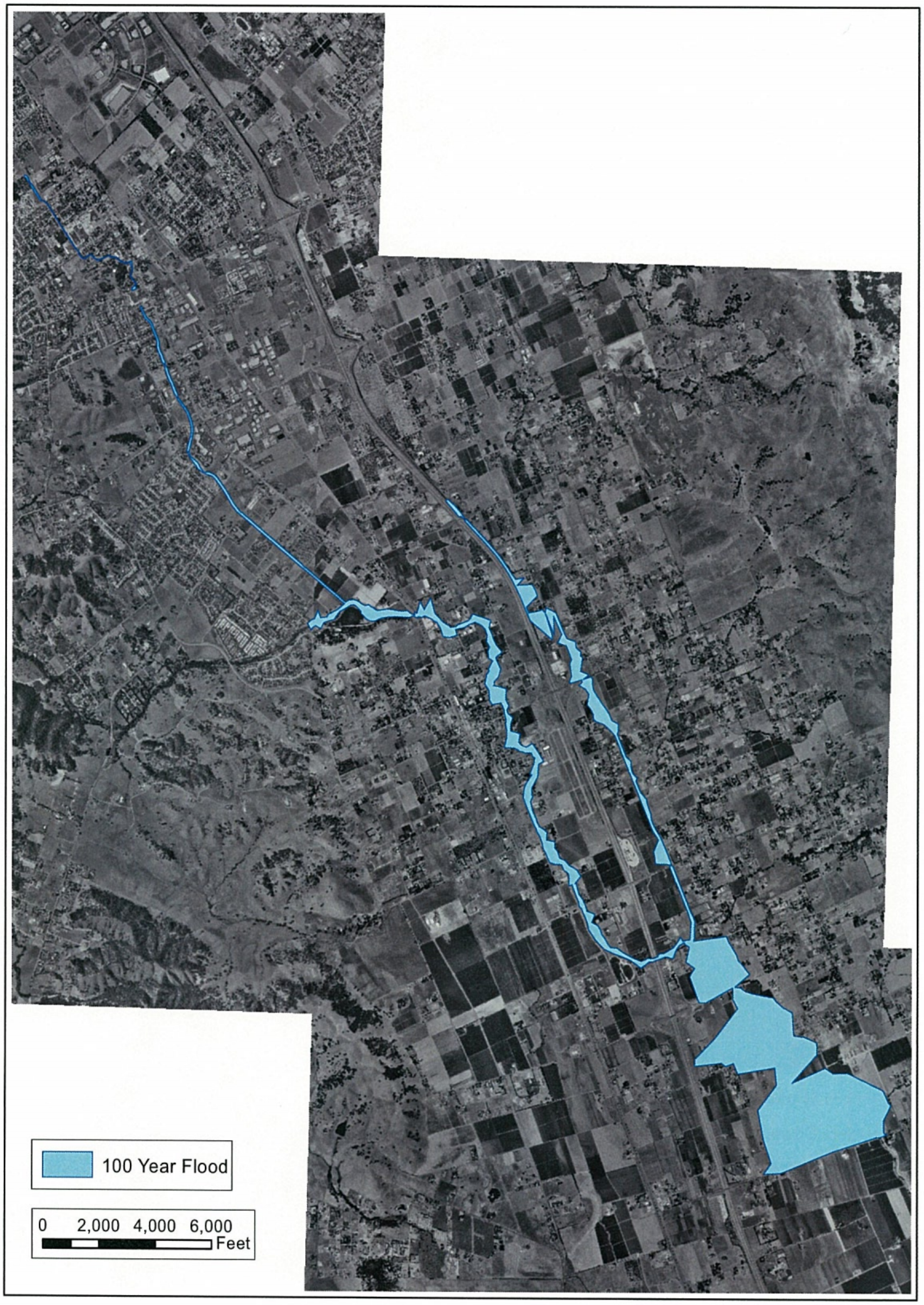


Figure 6-7. 100-year PME Inundation Boundaries in The Entire Reach

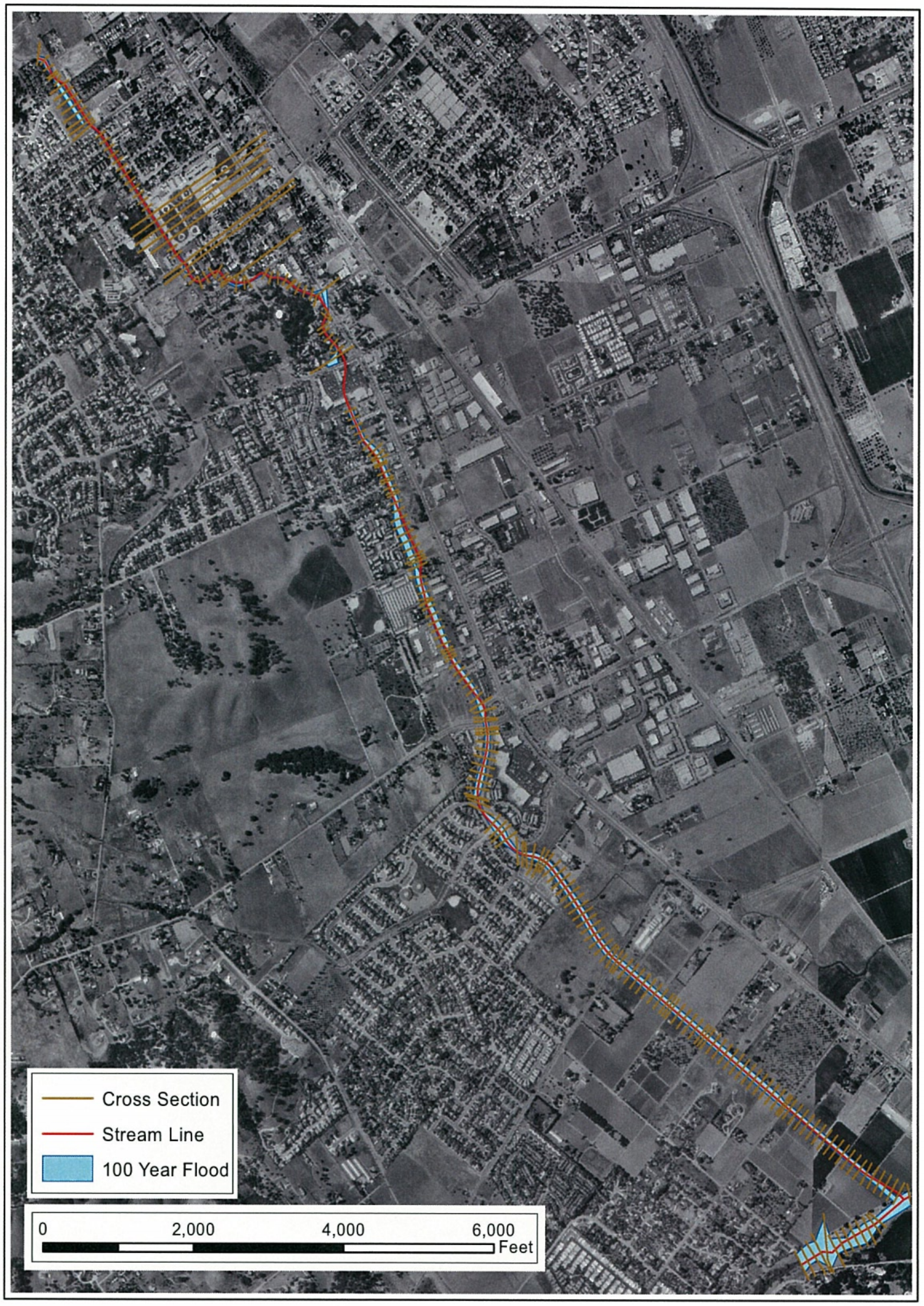


Figure 6-8. 100-year PME Inundation Boundaries in The Upper Reach

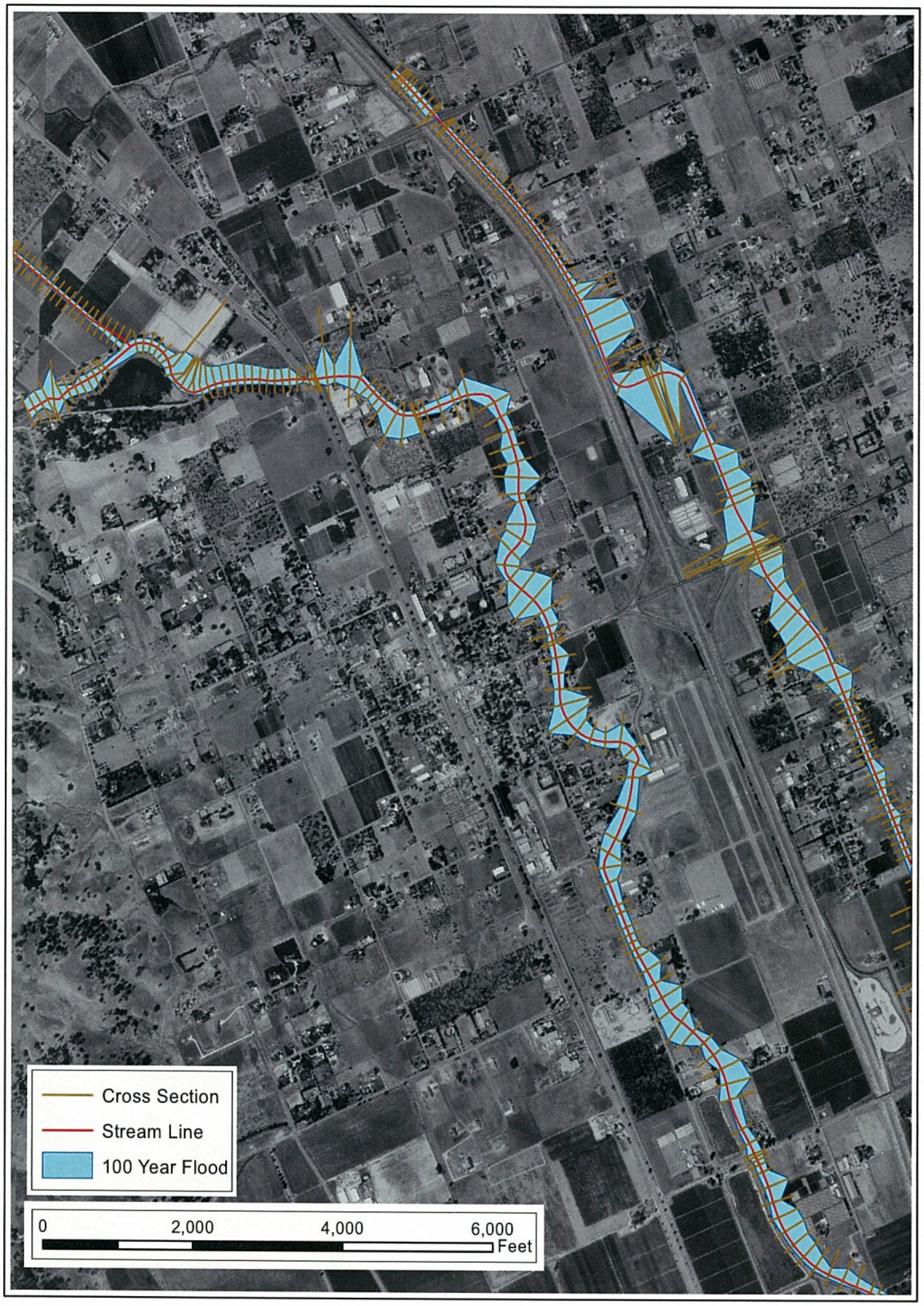


Figure 6-9. 100-year PME Inundation Boundaries in The Middle Reach

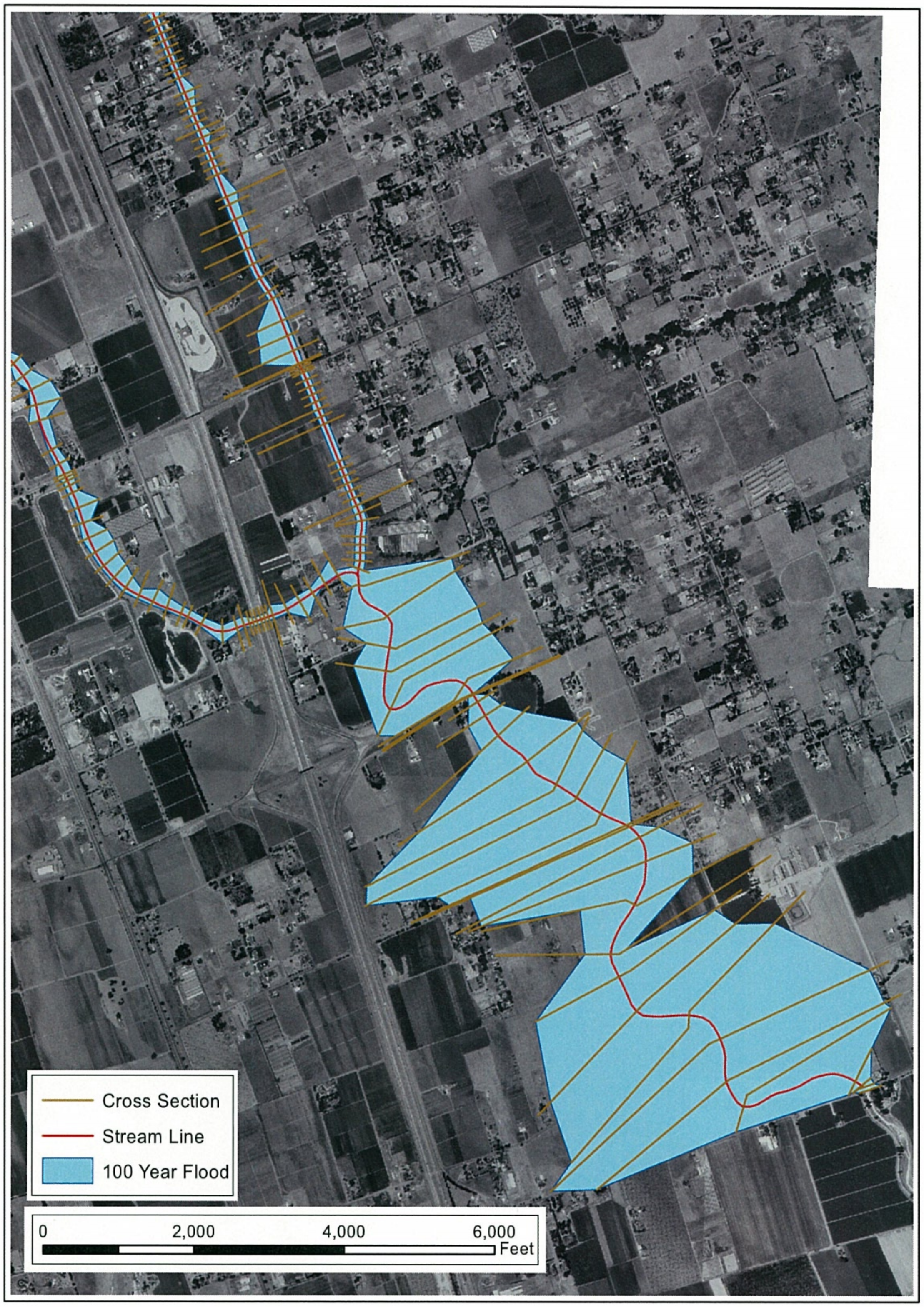


Figure 6-10. 100-year PME Inundation Boundaries in The Lower Reach

7.0 SUMMARY

This design study was conducted to update the preliminary channel design in Llagas Creek for the Upper Llagas Creek Flood Control Project. The sediment budget and potential morphological change within the study reaches were also assessed, based on the updated channel design.

A field reconnaissance investigation was initially conducted to examine the existing morphological conditions of the creek system. Bed material samples were taken at 16 locations within the study reaches, and a laboratory analysis was conducted to determine the sediment characteristics for the stable channel design, as shown in Figure 4-1. The entire study reaches were further divided into 16 sub-reaches based on the morphologic and hydrologic conditions. The starting river station of each sub-reach, which was used in the HEC-RAS model, and the hydrology under the with-project conditions are presented in Tables 3-2 and 3-3.

After evaluating several methodologies that are generally used to apply to the river channel design, the dominant (effective) discharge approach was applied to the low-flow channel design for the Llagas Creek system. The effective discharge for each sub-reach was derived, using the bed material load histogram method. Determination of the flow duration curve and the sediment-transport rating curve was made via the SAM computer program. The derived reach specific effective discharges are listed in Table 5-1. The estimated effective discharges are much smaller than the 2-year peak flow discharges.

The stable channel analytical design method in SAM was used to size the stable low-flow channel for ten of the sixteen sub-reaches. The preliminary channel design for other six sub-reaches (four sub-reaches in Reach 8, R7S4, and R6S2) was not modified, because of the project constrain or the environmental considerations. The SAM stable channel analysis indicated that the stable low-flow channel could be determined in five sub-reaches (i.e., R7S3, R6S1, R5, R14S4, & R14S2). The preliminarily designed low-flow channel dimensions were also modified for other three sub-reaches (R7S2, R7S1 & R4) for the purpose of being consistent with the dimensions or flow capacity of the adjacent low-flow channels. The updated low-flow channel dimensions are presented in Table 5-2.

The updated channel design was compared to the preliminary design, as shown in Figures 5-62 to 5-69. Except for Reach 4 where the channel is significantly widened in this updated design, the bottom widths of the newly designed low-flow channels are similar to the preliminary design. However, the updated bench elevations are lower than the preliminary design.

The sediment budget was assessed under the average annual condition, effective discharge scenario, ten-percent and one-percent chance exceedance flood events, as listed in Tables 5-7 to 5-10. The sediment budget is likely to be balanced for Reach 8, upper segment of Reach 7, R6S1, & R14S2, resulting in the quasi-stable channels under the four flow scenarios. However, the sediment supply will significantly exceed the sediment yield in the lower segment of Reach 7 including the diversion channel and in Reach 4. Consequently, significant sedimentation is expected to occur in these sub-reaches (reaches). Channel erosion will marginally occur in Reach 5 and Su-breach R14S3, as the sediment yield estimated in these two channel segments is greater than the sediment supply delivered from the upstream segment.

The diversion channel under the alignment formulated in the preliminary design is not stable. The abruptly flattened channel longitudinal gradient of the proposed diversion channel will result in significant sedimentation. The sedimentation problem may be mitigated to achieve a stable channel condition by re-grading or re-aligning the preliminarily designed diversion channel that would lead to a steeper channel gradient.

Floodplain mapping was performed to determine the inundation stages and floodplain boundaries under the modified channel conditions for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year annual peak flows. In general, flood stages exceed the capacity of the main channels, particularly in the lower reach (Reach 4) for the return flood events of 100-, 250-, and 500-year flows. For a flow event that has a recurrence interval of less than 100-years, floodwaters are mostly contained within the main channel and the floodplain is confined to the channel cross sections only.

8.0 REFERENCES

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APPENDIX A

Site Photographs From Field Reconnaissance

July 23-24, 2006

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Photo 76. Grouted Rocks under Leavesley Road Bridge.
Photo 77. Llagas Creek below Leavesley Road Bridge

West Little Llagas Creek
(from upstream to downstream)

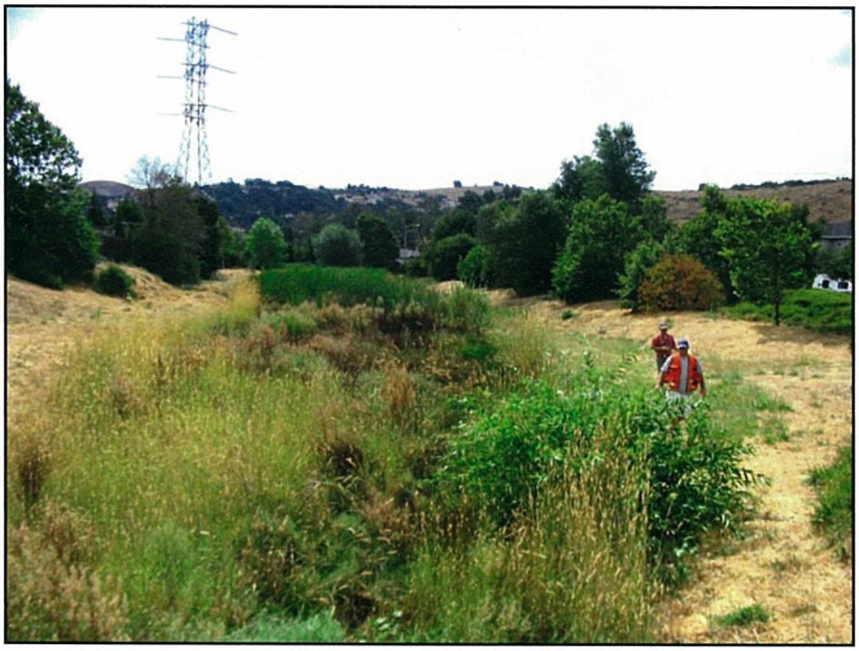


Photo 1. West Little Llagas Creek Upstream of Llagas Road
View upstream. Note dense in-channel vegetation and cleaned floodplain areas.

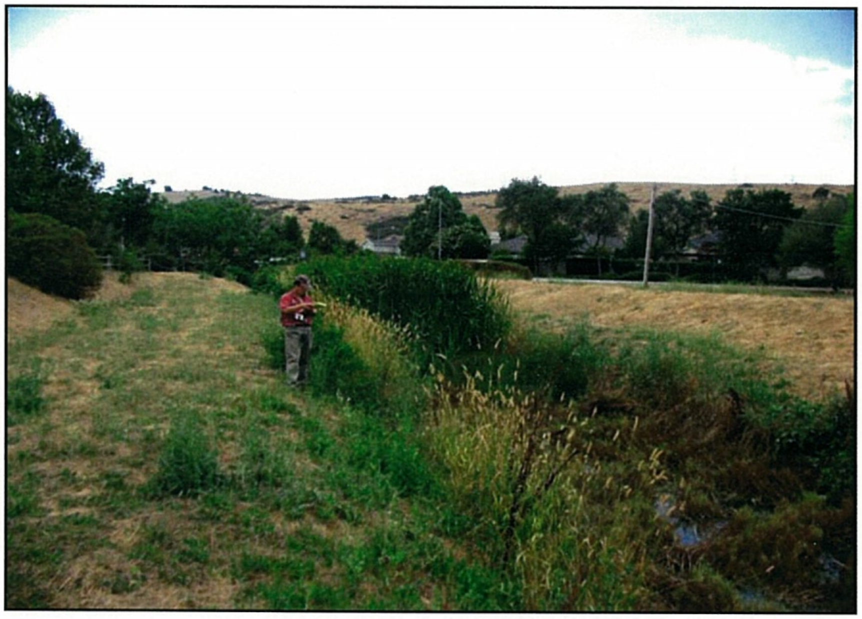


Photo 2. West Little Llagas Creek Downstream of Llagas Road
View upstream. Note alternating open and vegetated channel reaches and clean floodplain areas.

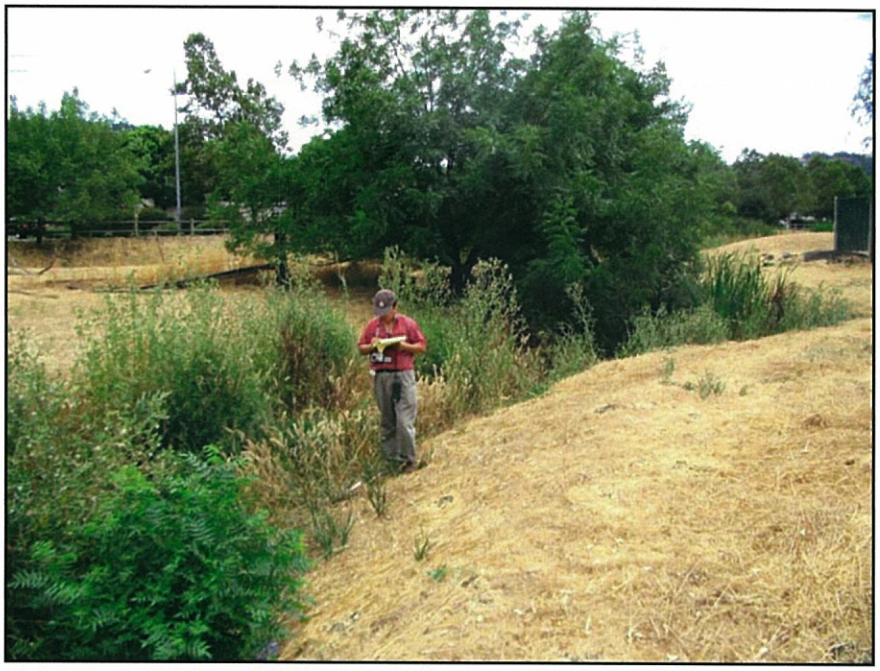


Photo 3. West Little Llagas Creek Downstream of Llagas Road
View upstream. Note dense in-channel vegetation.

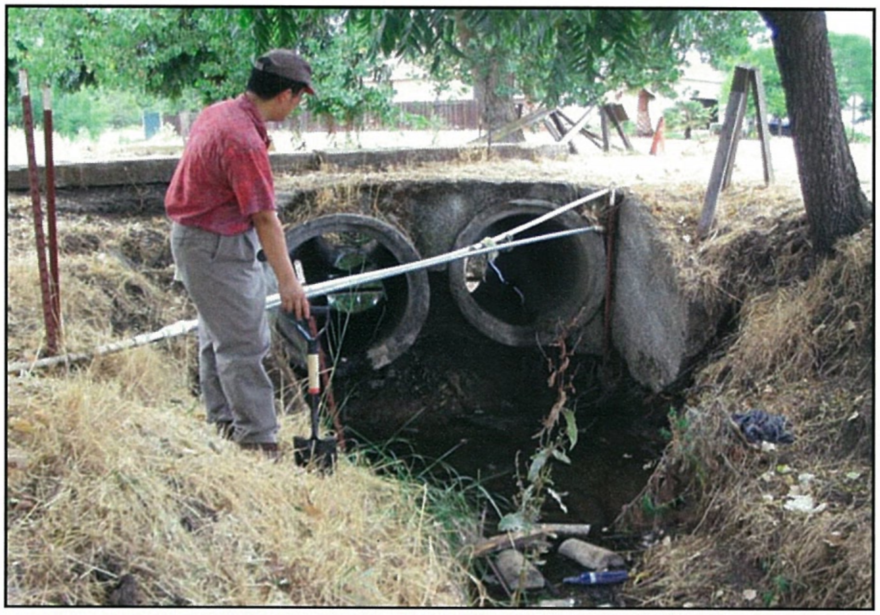


Photo 4. Culvert on West Little Llagas Creek under Private Road
View upstream. Channel downstream of culvert was artificially deepened for flood control purposes, which caused bed erosion at culvert by about 3 ft. Note litter and rubble in creek channel.

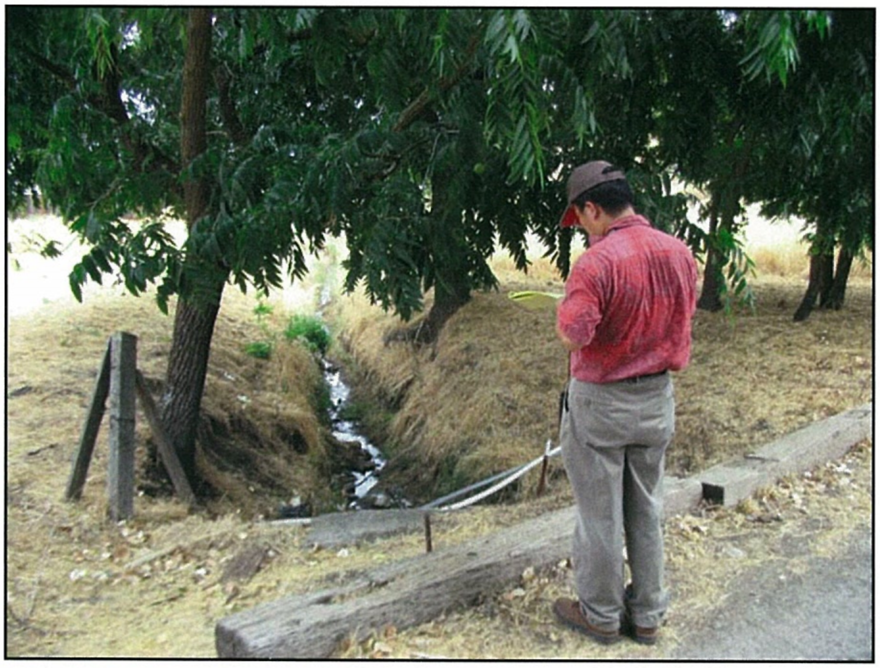


Photo 5. West Little Llagas Creek below Private Road
View downstream. Channel width is 3-5 ft. According to local resident, channel in this reach was deepened and relocated south.

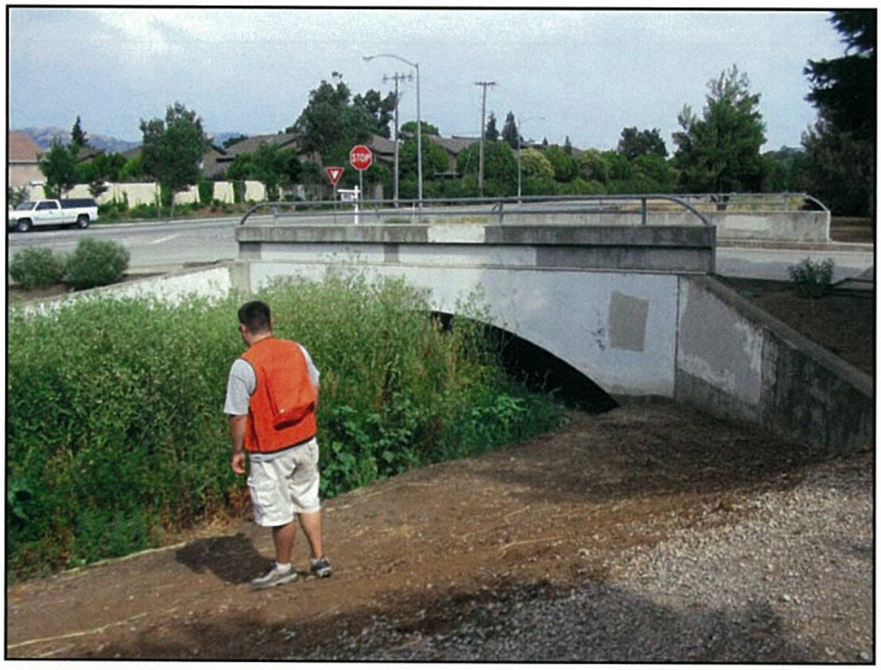


Photo 6. West Little Llagas Creek at Hillwood Lane
View downstream. Project reach begins at this location. Note dense in-channel vegetation.

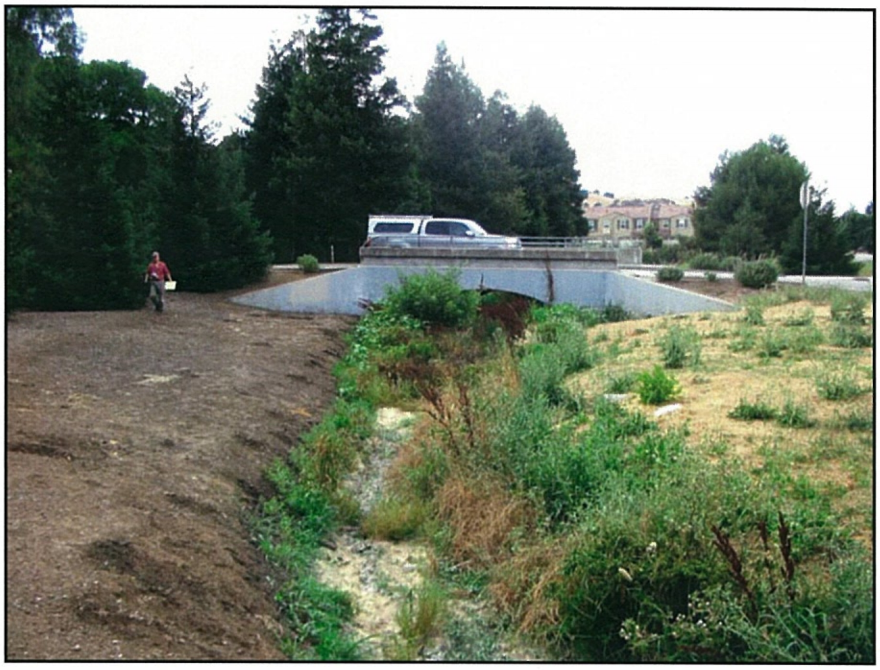


Photo 7. West Little Llagas Creek Downstream of Hillwood Lane
View upstream.

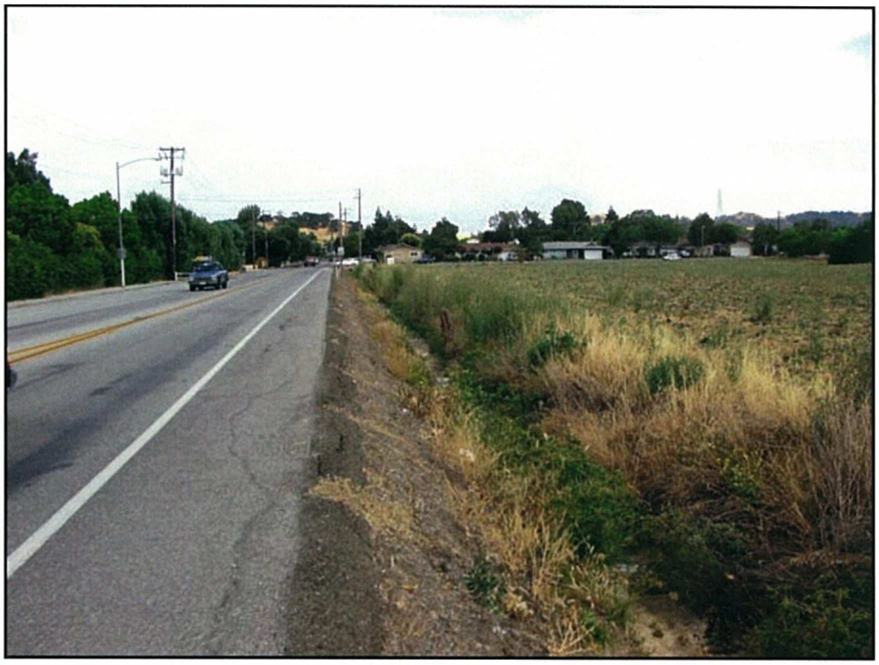


Photo 8. West Little Llagas Creek Upstream of Wright Avenue
View downstream along Hale Avenue.

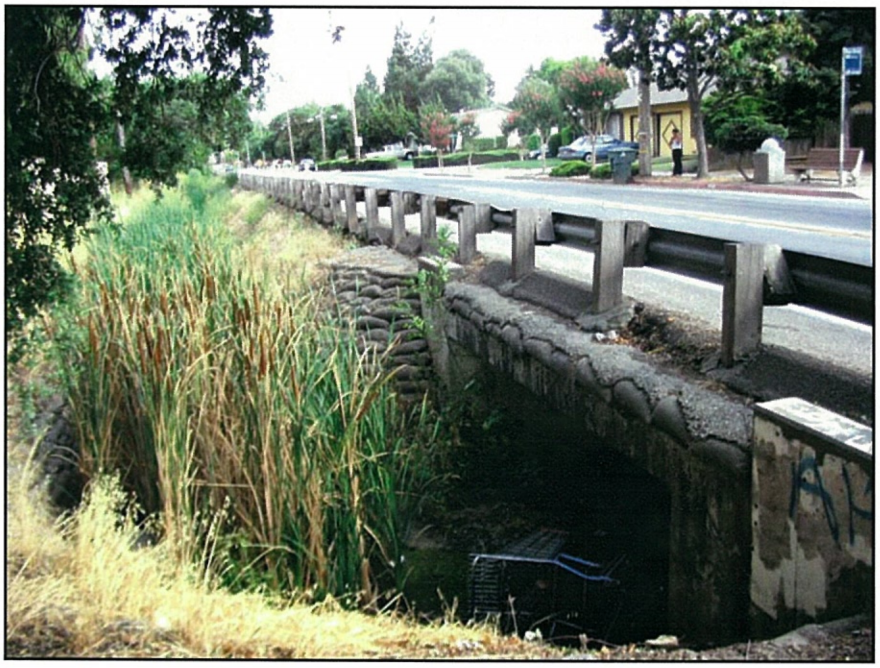


Photo 9. West Little Llagas Creek Downstream of Wright Avenue Culvert
View downstream. Note dense in-channel vegetation and bank protection works.

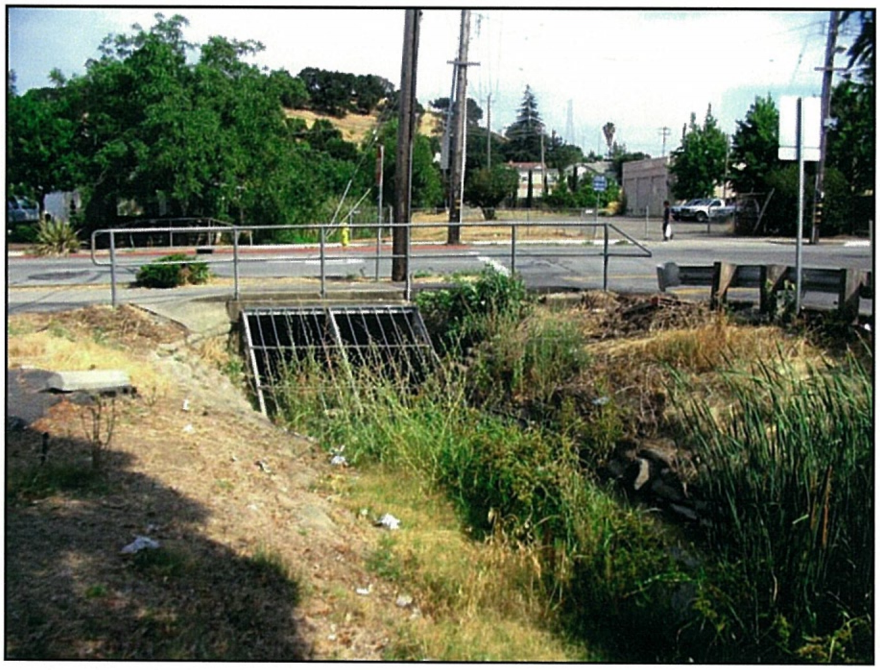


Photo 10. West Little Llagas Creek at West Main Avenue Culvert
View downstream. Note backwater upstream of culvert and bank protection works.

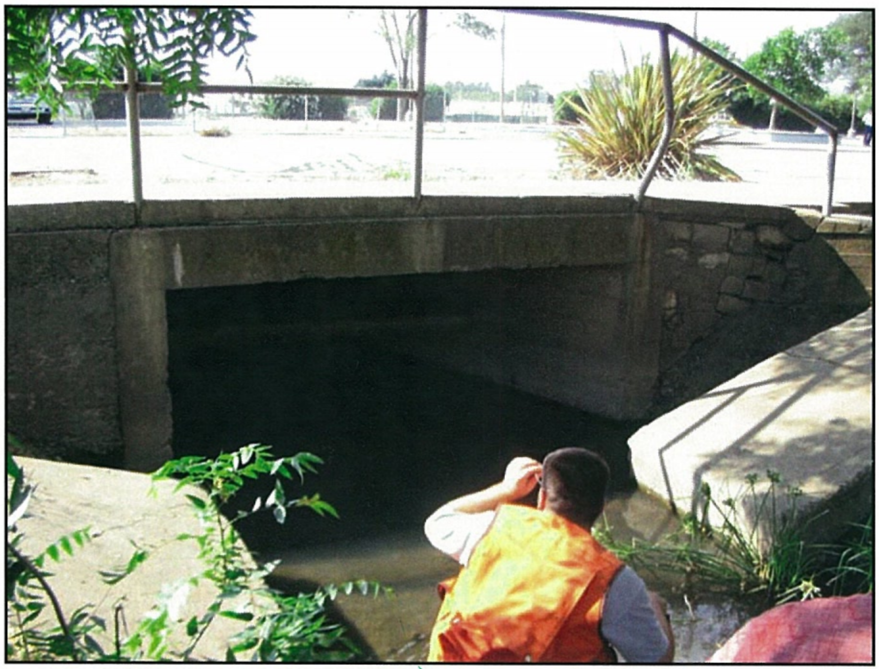


Photo 11. Exit of West Main Avenue Culvert

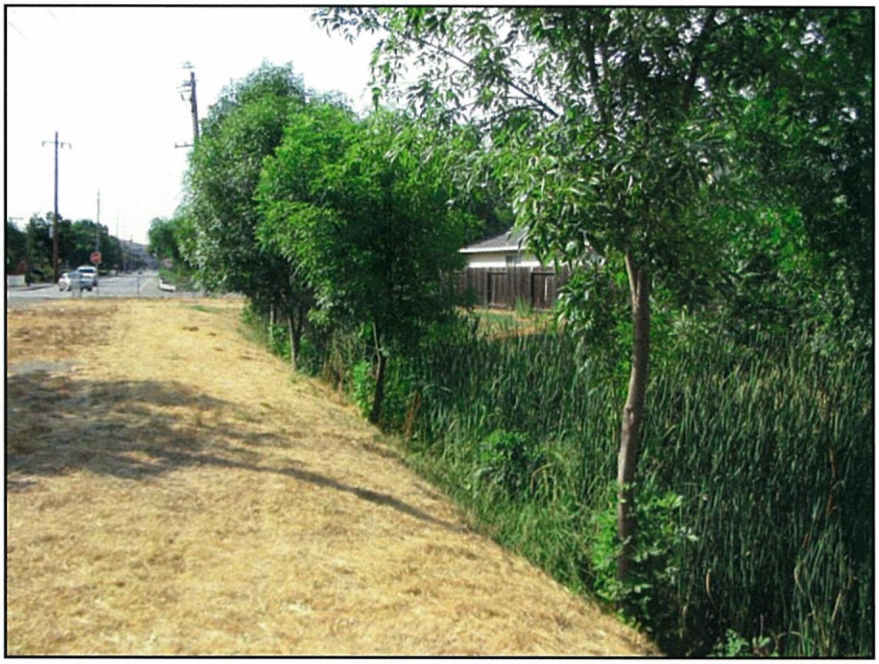


Photo 12. West Little Llagas Creek Downstream of West Main Avenue
View upstream. Note heavily vegetated channel and clean banks.

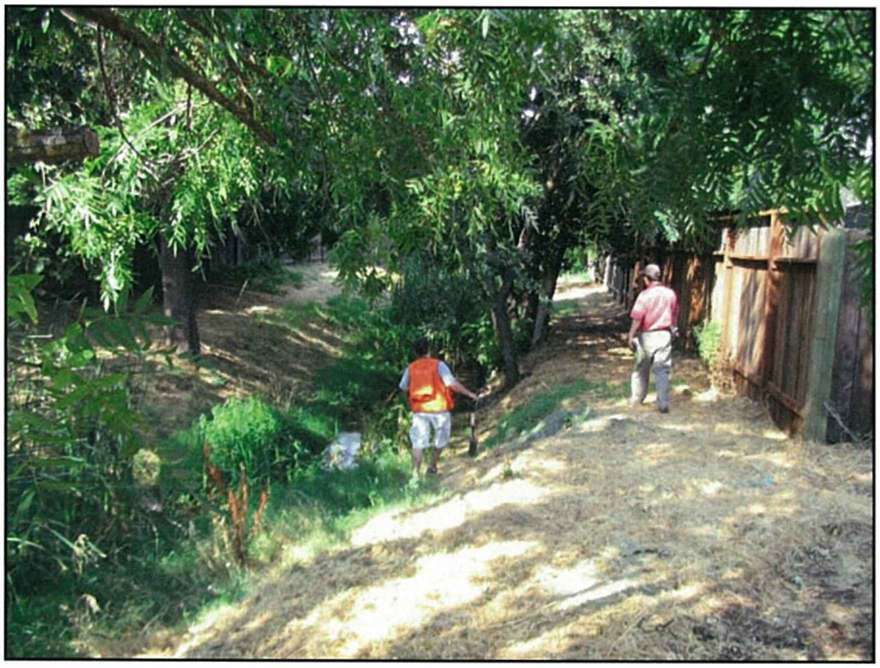


Photo 13. West Little Llagas Creek between West Main Avenue and Del Monte Avenue
View downstream.

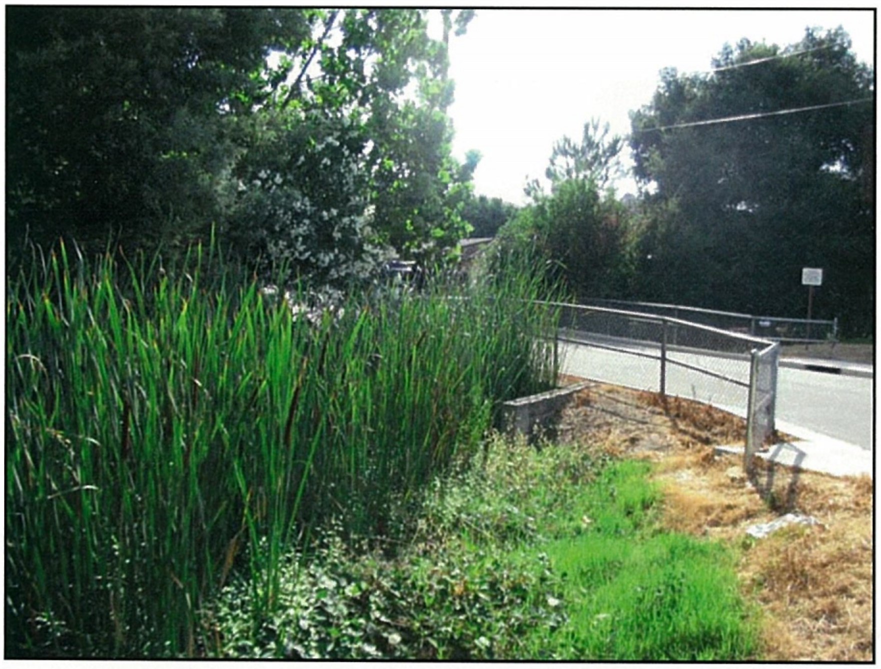


Photo 14. West Little Llagas Creek Downstream of Warren Avenue Culvert
Note heavily vegetated channel.

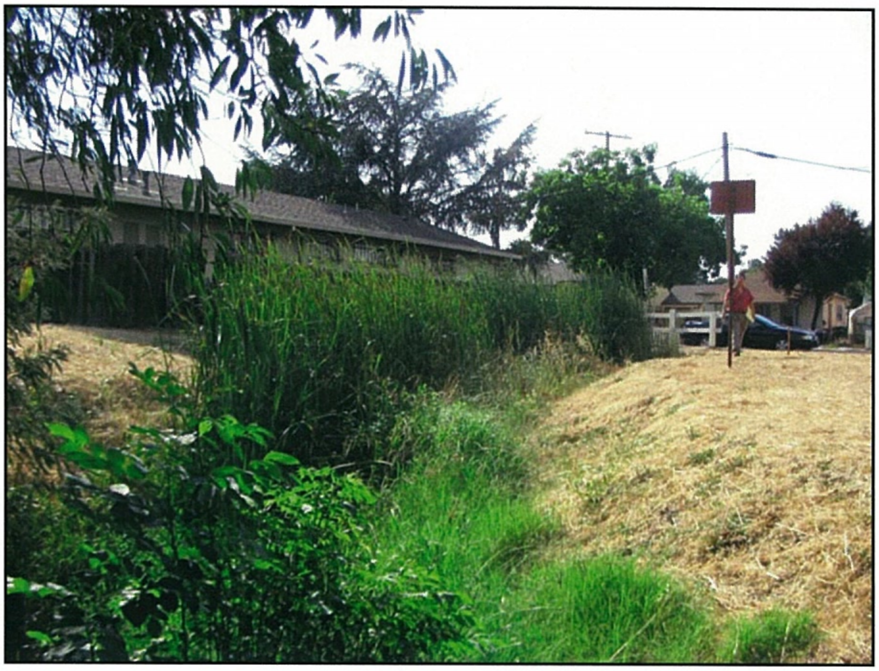


Photo 15. West Little Llagas Creek Downstream of West 2nd Street
View upstream towards culvert. Note dense in-channel vegetation.

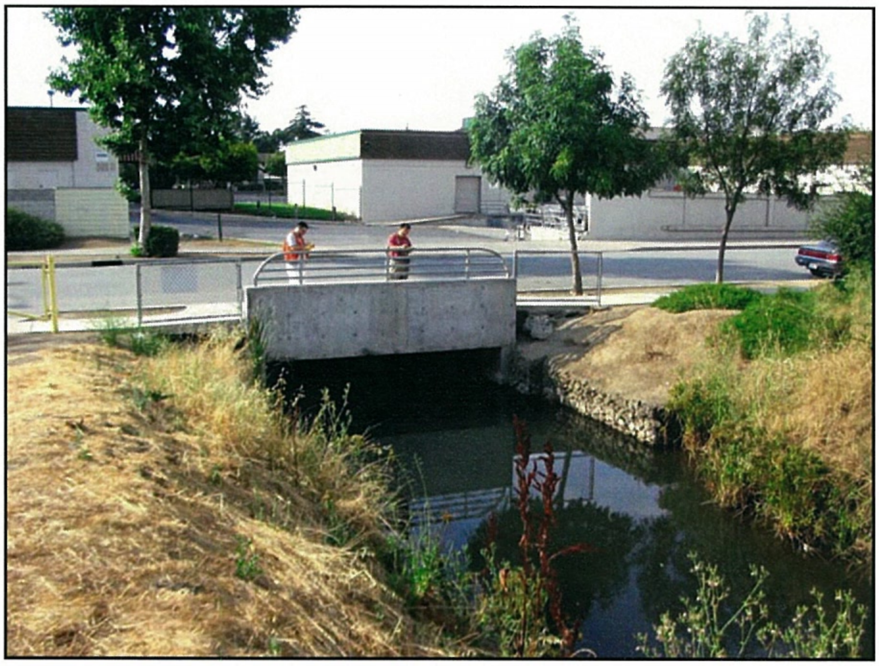


Photo 16. West Little Llagas Creek at Ciolino Avenue
View upstream toward culvert under shopping center.

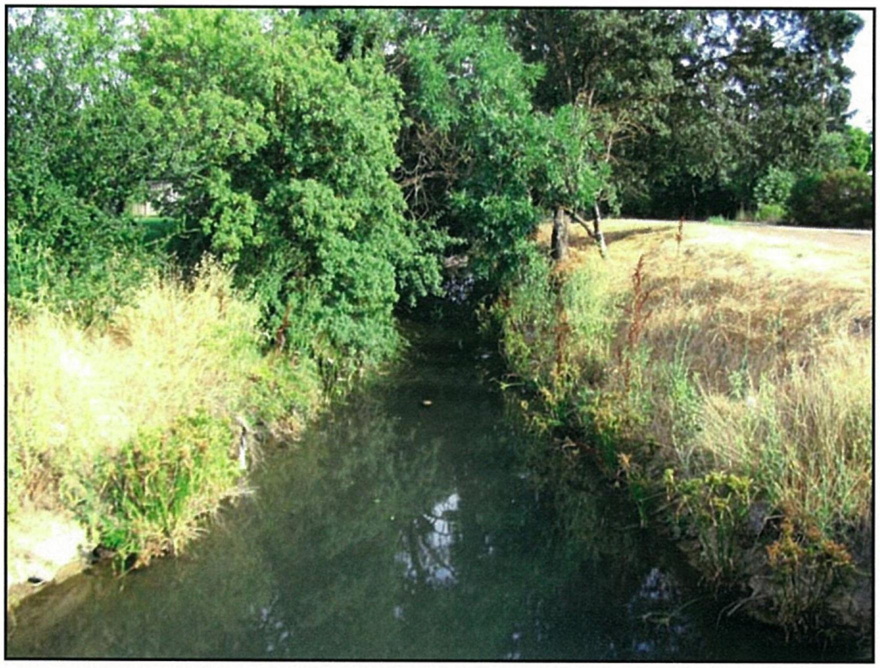


Photo 17. West Little Llagas Creek below Ciolino Avenue
View downstream.

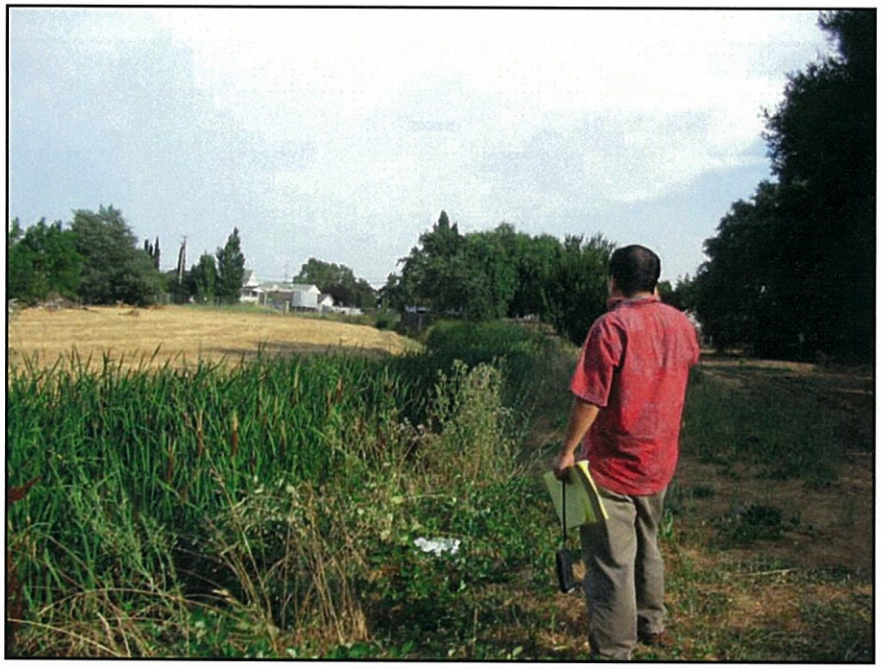


Photo 18. West Little Llagas Creek Upstream of Spring Avenue
View downstream. Note dense in-channel vegetation and clean floodplain.

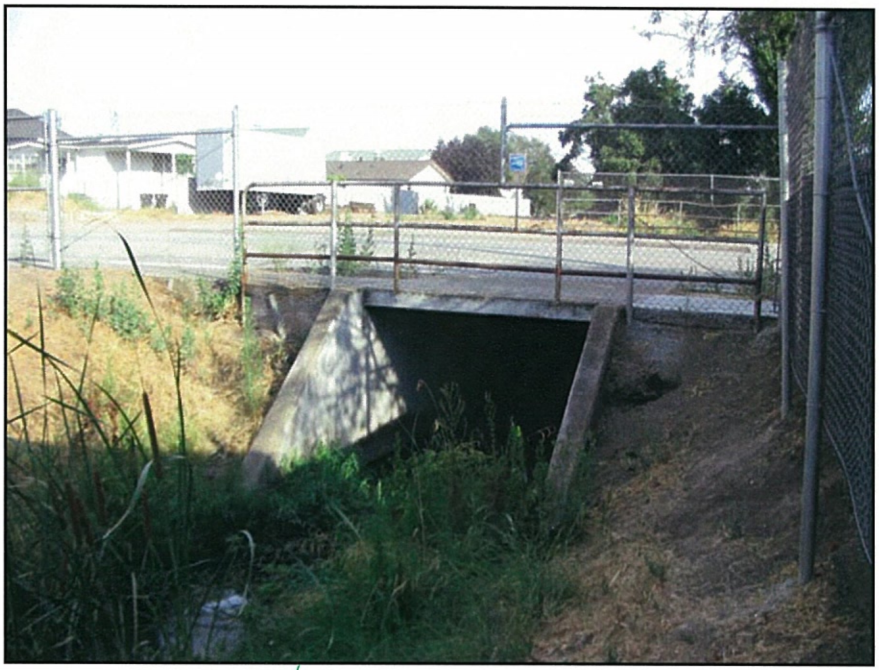


Photo 19. Entrance to Spring Avenue Culvert on West Little Llagas Creek
View downstream.

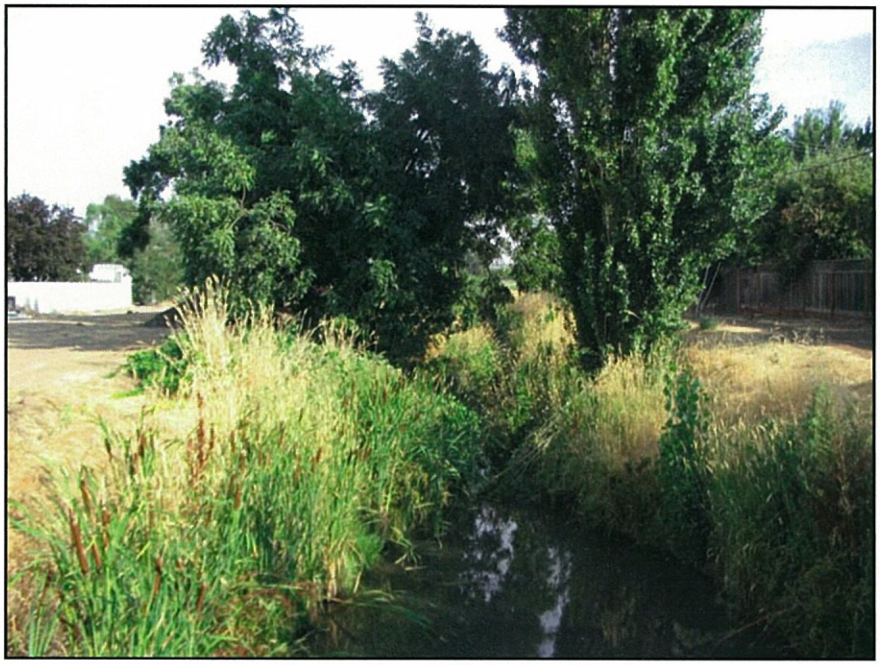


Photo 20. West Little Llagas Creek below Spring Avenue
View downstream. Note mature trees along creek banks.

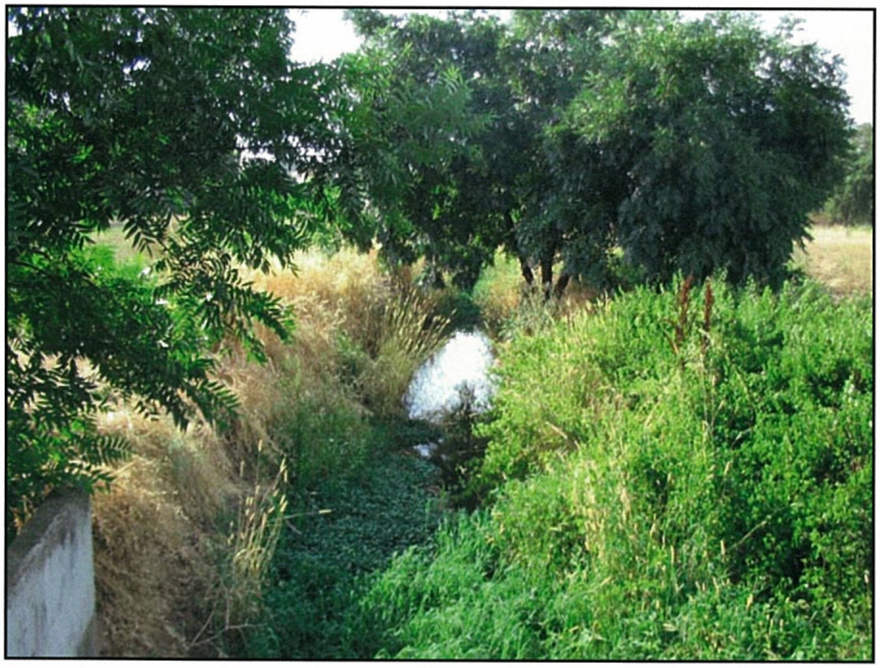


Photo 21. West Little Llagas Creek above Cosmo Avenue
View upstream.

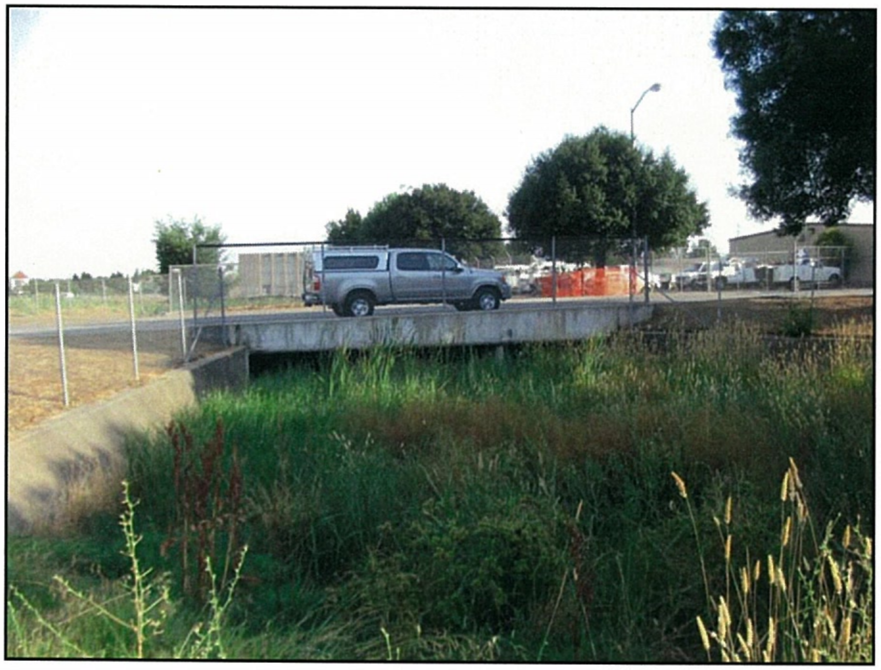


Photo 22. West Little Llagas Creek at Edes Court Culvert
View downstream. Note dense vegetation in creek channel.

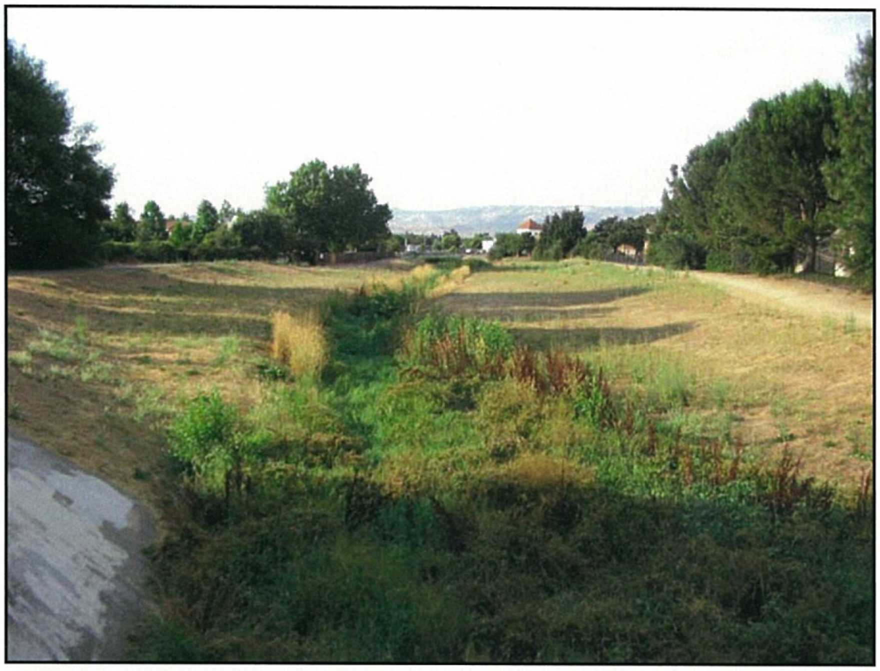


Photo 23. West Little Llagas Creek below West Edmundson Avenue
View downstream.

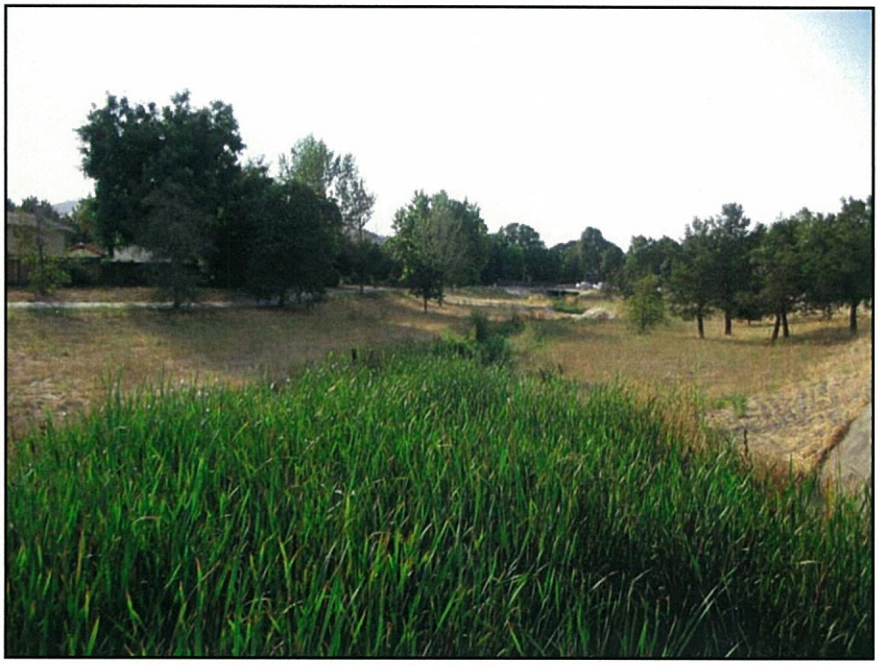


Photo 24. West Little Llagas Creek above South La Crosse Drive
View upstream. Note dense in-channel vegetation and cleaned floodplain areas.

Photo 25. West Little Llagas Creek Diversion Channel
View upstream toward confluence with West Little Llagas Creek.

Photo 26. Downstream End of West Little Llagas Creek Diversion Channel
at Watsonville Road.

Photo 27. Reach of West Little Llagas Creek Running Parallel to Watsonville Road
View upstream. Channel in this reach is small and heavily vegetated.

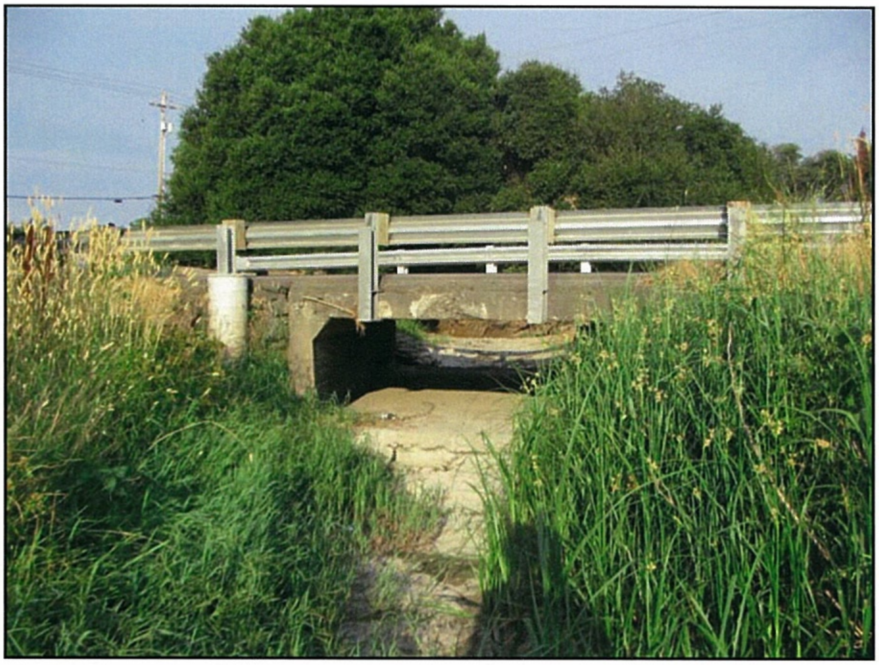


Photo 28. Entrance to Watsonville Road Culvert on West Little Llagas Creek
View downstream.

East Little Llagas Creek
(from upstream to downstream)

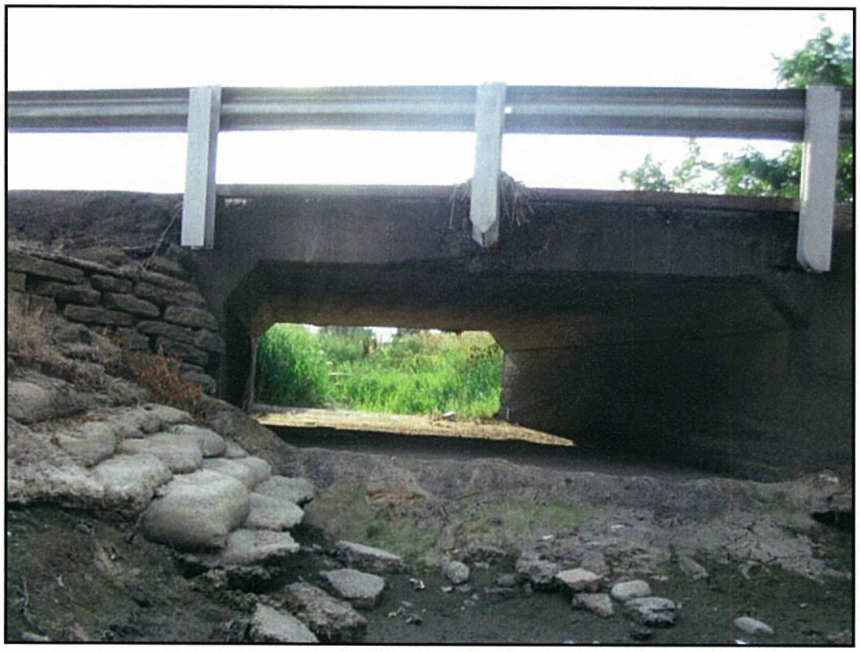


Photo 29. Exit of Watsonville Road Culvert
View upstream. Note debris deposited on top of road crossing after recent floods and erosion along toe of bank protection works.

Photo 30. East Little Llagas Creek below Watsonville Road
View upstream. Bed material is composed of gravel.

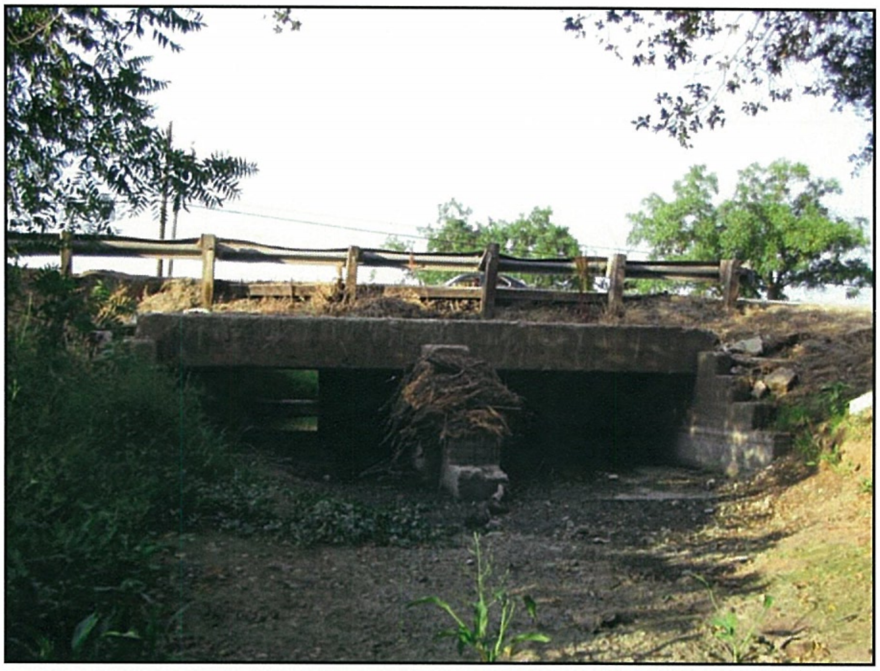


Photo 31. Entrance to Monterey Road Culvert on East Little Llagas Creek
View downstream. Note debris deposited by recent floods.

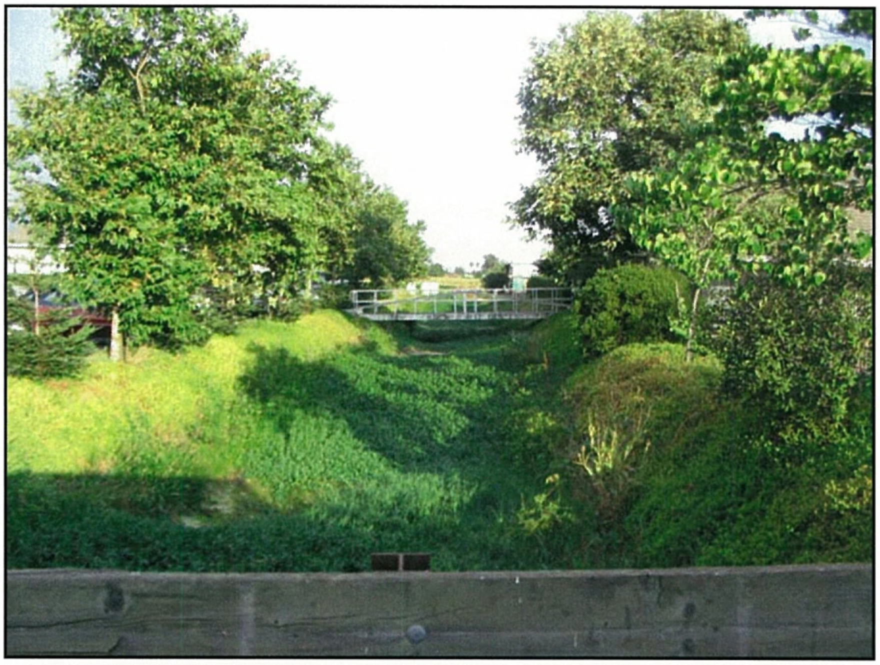


Photo 32. East Little Llagas Creek below RV Park Bridge #2
View downstream.

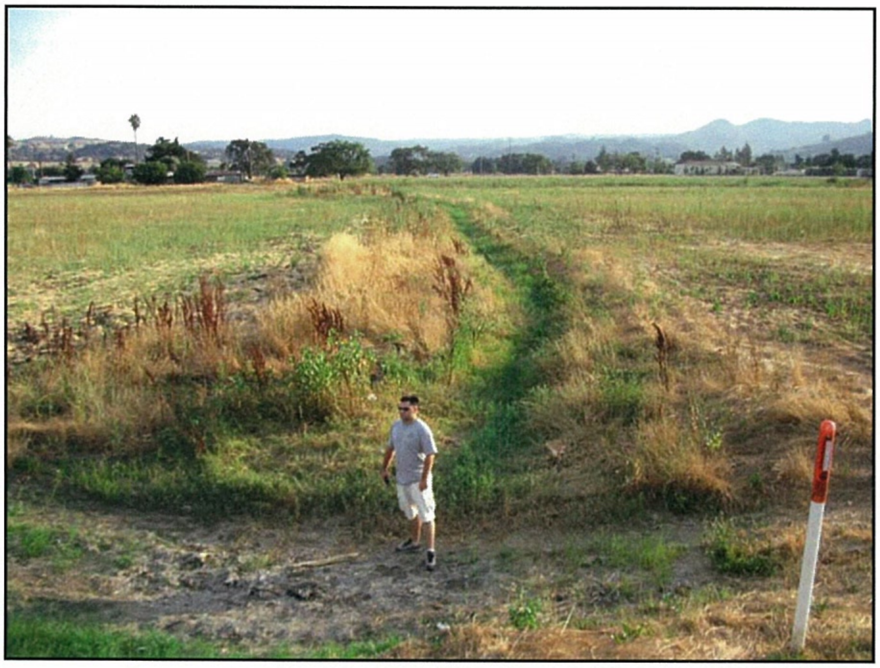


Photo 33. East Little Llagas Creek upstream of South Pacific Railroad View upstream. Note small size of channel.

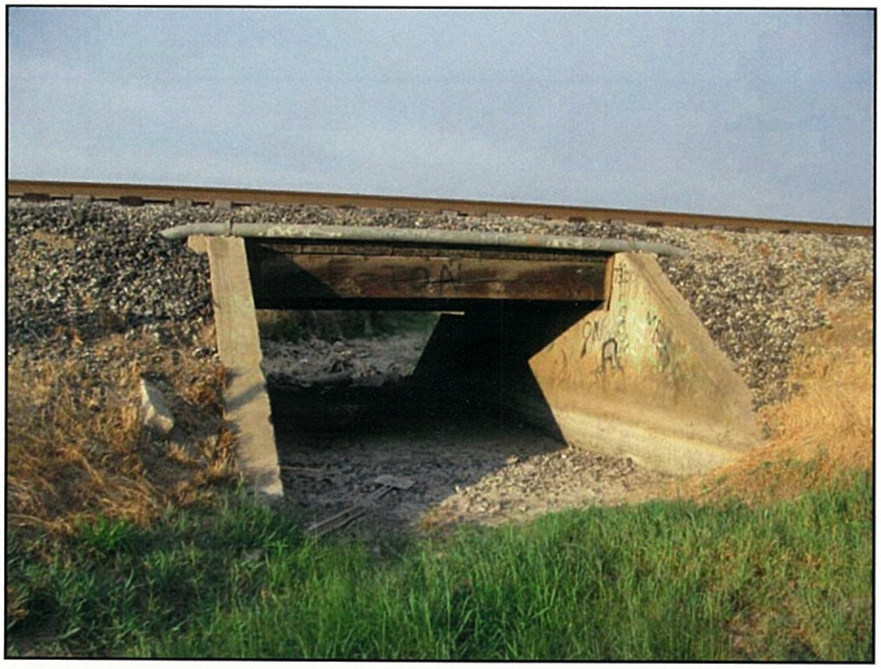


Photo 34. Entrance to South Pacific Railroad Culvert on East Little Llagas Creek View downstream.

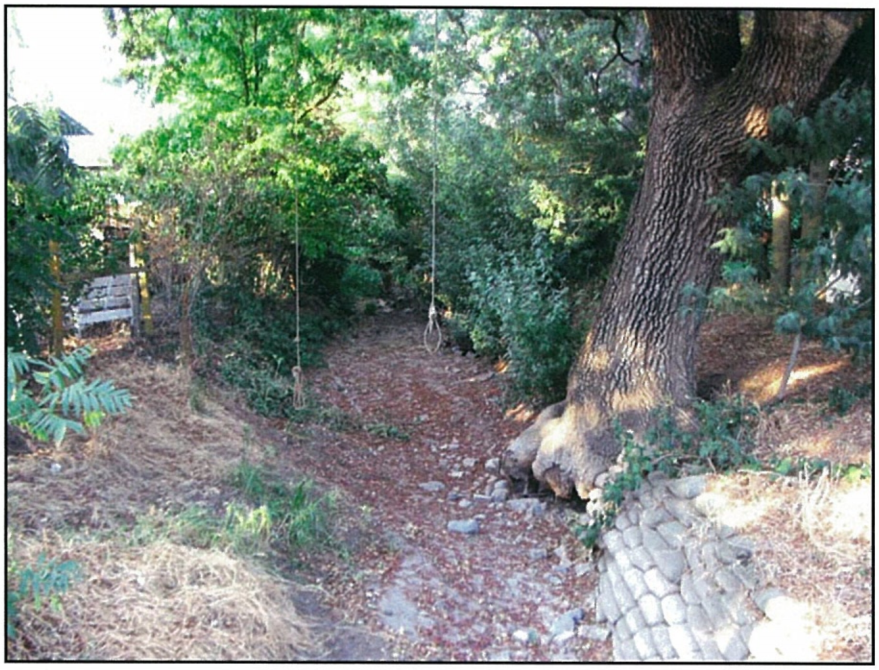


Photo 35. East Little Llagas Creek below Llagas Avenue Culvert
View downstream. Note exposed tree roots.

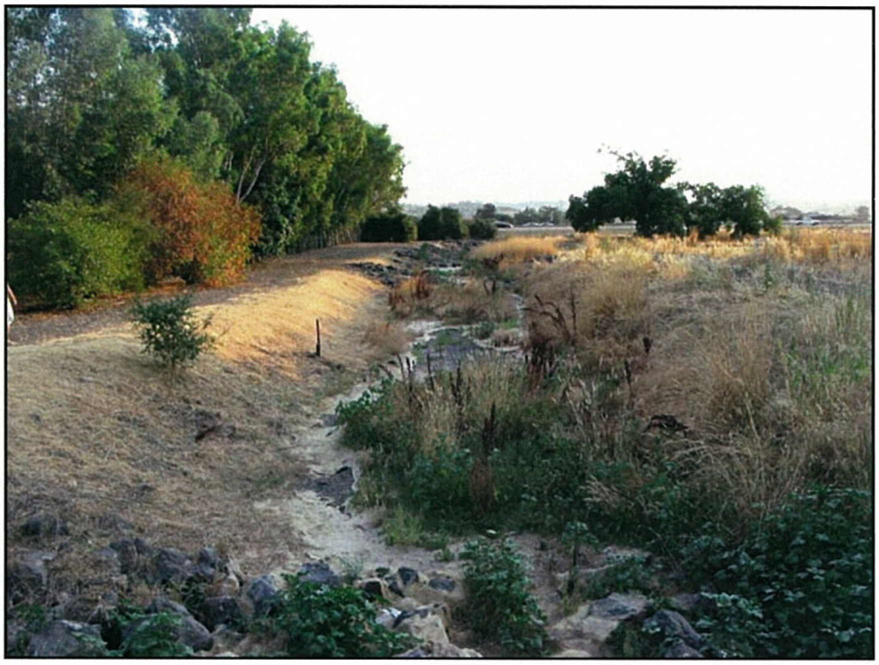


Photo 36. East Little Llagas Creek above Sycamore Avenue
View upstream. Note bank protection at channel bend.

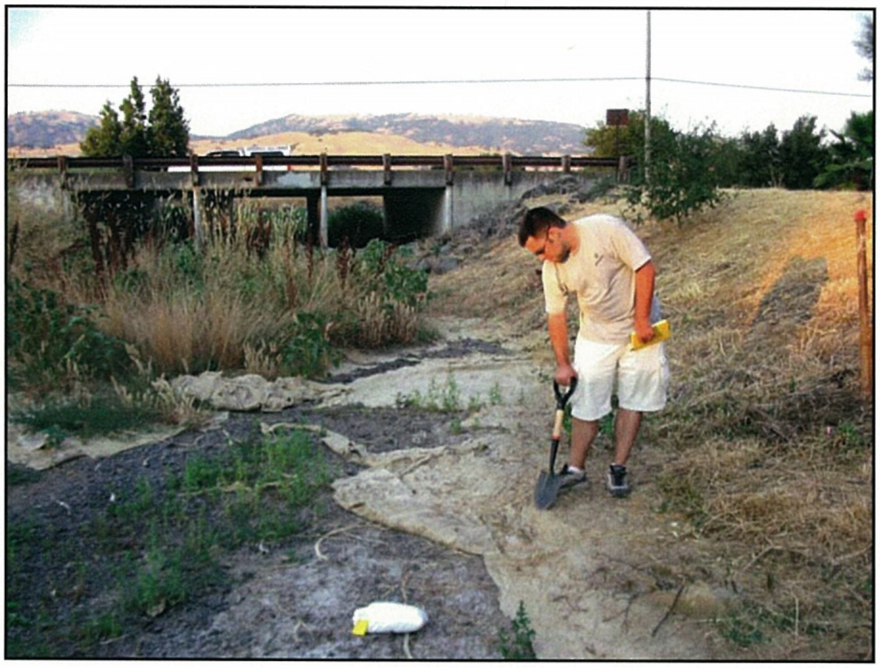


Photo 37. East Little Llagas Creek at Sycamore Avenue
View downstream. Stream bed is composed of fine gravel and partially covered with dried biomass (algae).

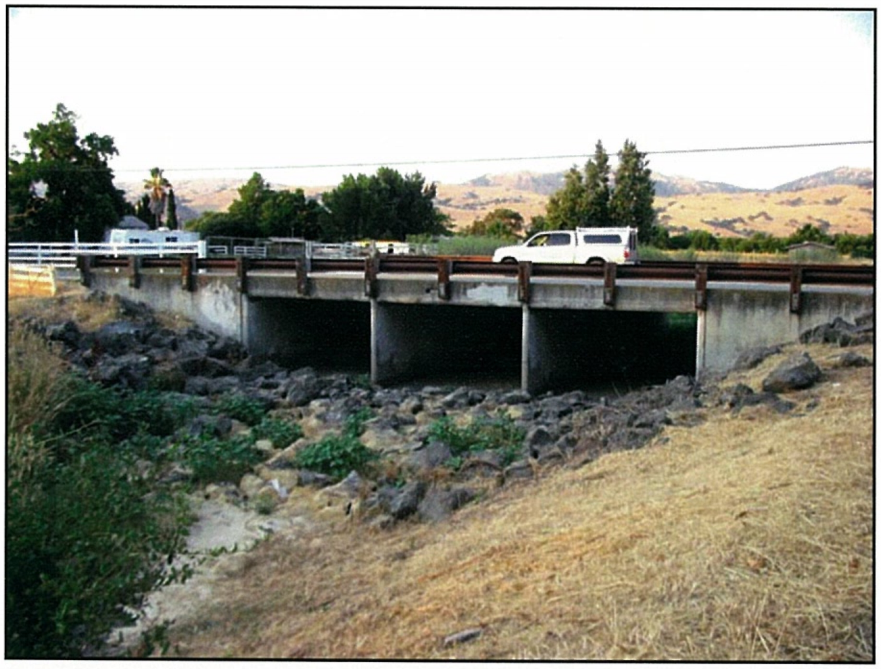


Photo 38. Entrance to Sycamore Avenue Culvert on East Little Llagas Creek
View downstream. Channel at culvert is protected with rock revetment.

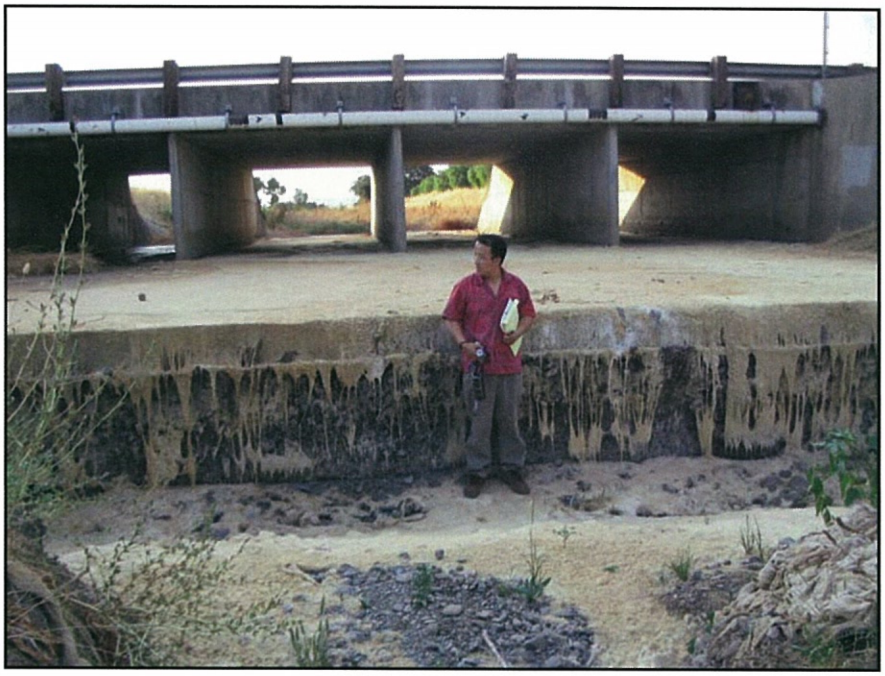


Photo 39. Drop Structure downstream of East San Martin Avenue Culvert on East Little Llagas Creek
View upstream. Note bed scour immediately downstream of drop structure.

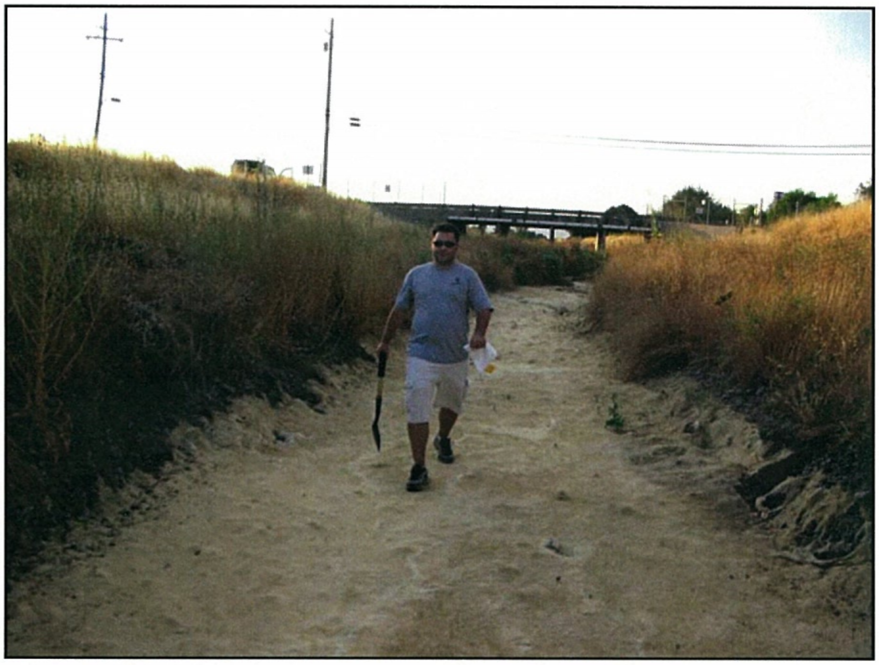


Photo 40. East Little Llagas Creek below East San Martin Avenue
View upstream. Note dry biomass (algae) completely covering gravel stream bed.

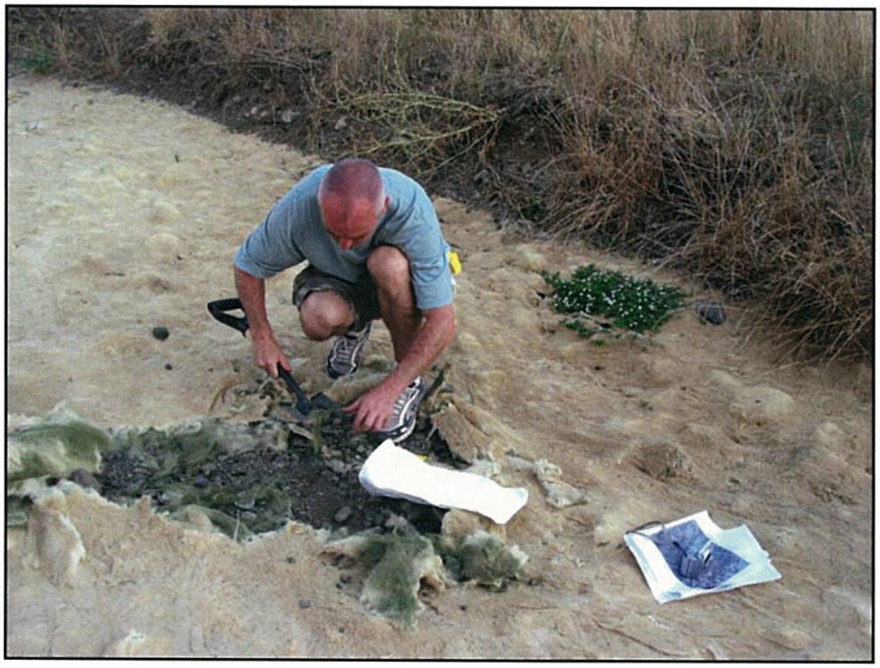


Photo 41. Stream Bed in East Little Llagas Creek below East San Martin Avenue

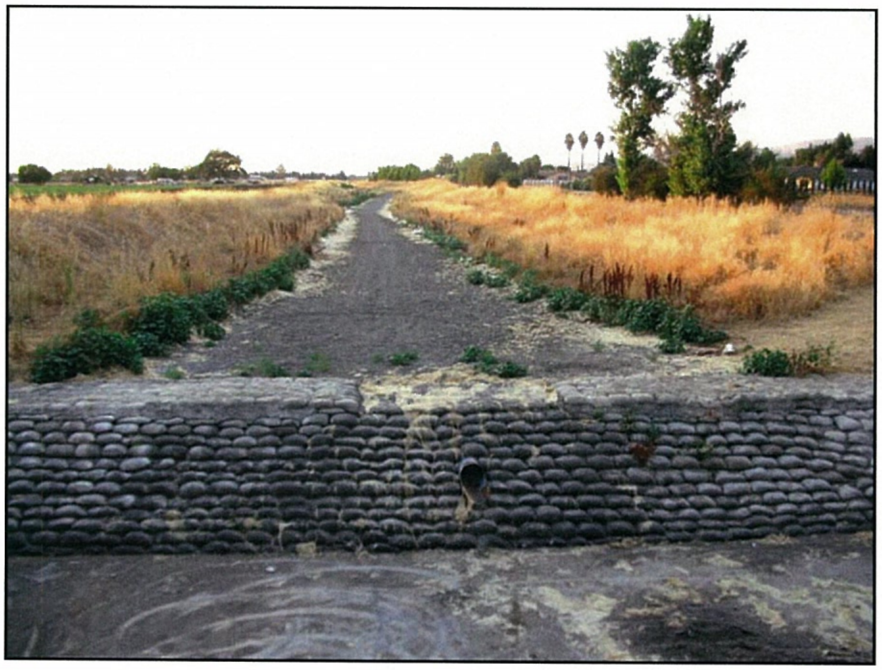


Photo 42. Drop Structure on East Little Llagas Creek Immediately Upstream of Church Avenue Culvert
View upstream.

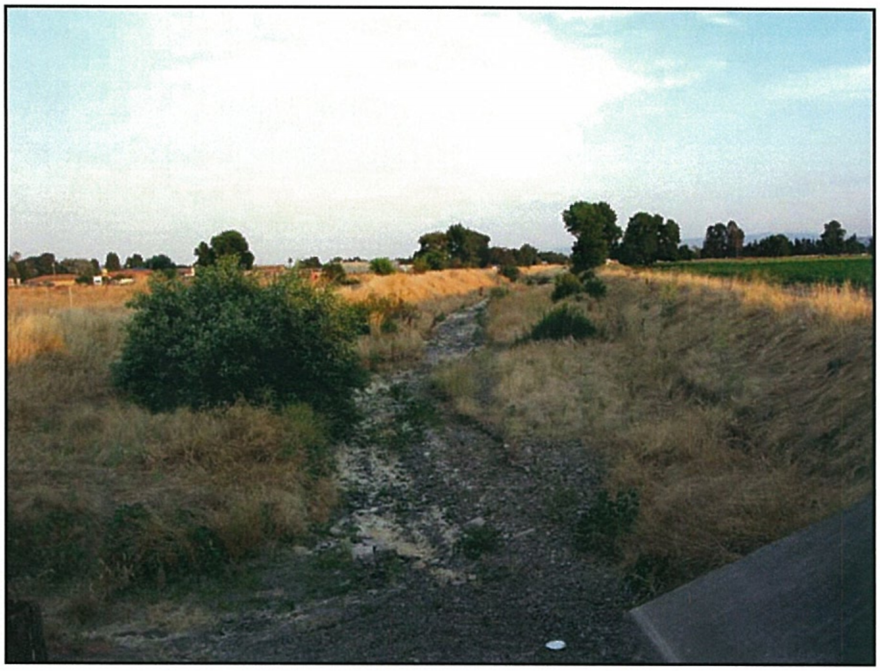


Photo 43. East Little Llagas Creek below Church Avenue
View downstream.

Photo 44. Sampling of Bed Material in East Little Llagas Creek below Church Avenue

Llagas Creek
(from upstream to downstream)

07/24/2006 10:22

Photo 45. Chesbro Reservoir

07/24/2006 10:30

Photo 46. Access Road Across Llagas Creek Downstream of Chesbro Reservoir

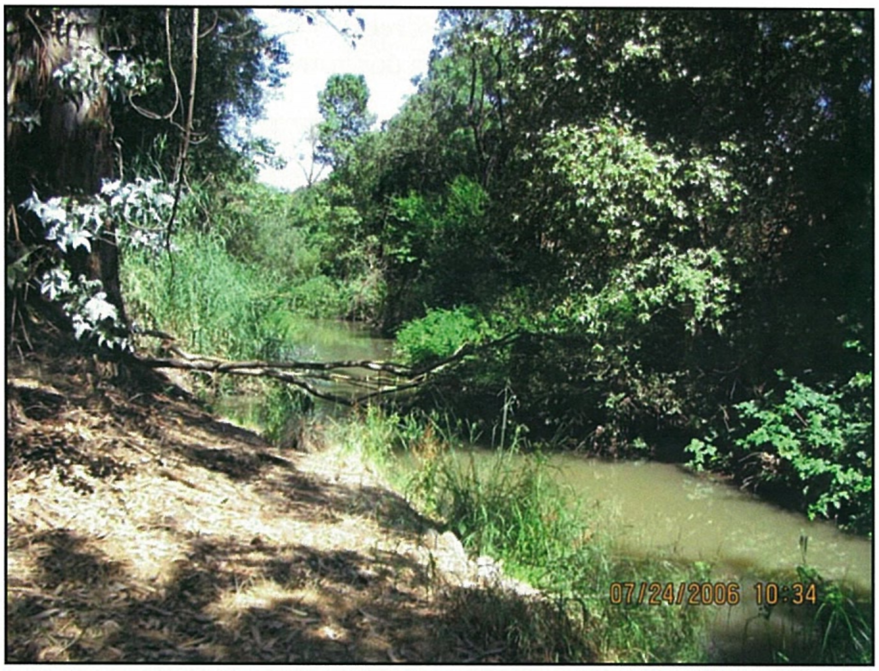


Photo 47. Llagas Creek below Chesbro Reservoir. View upstream.

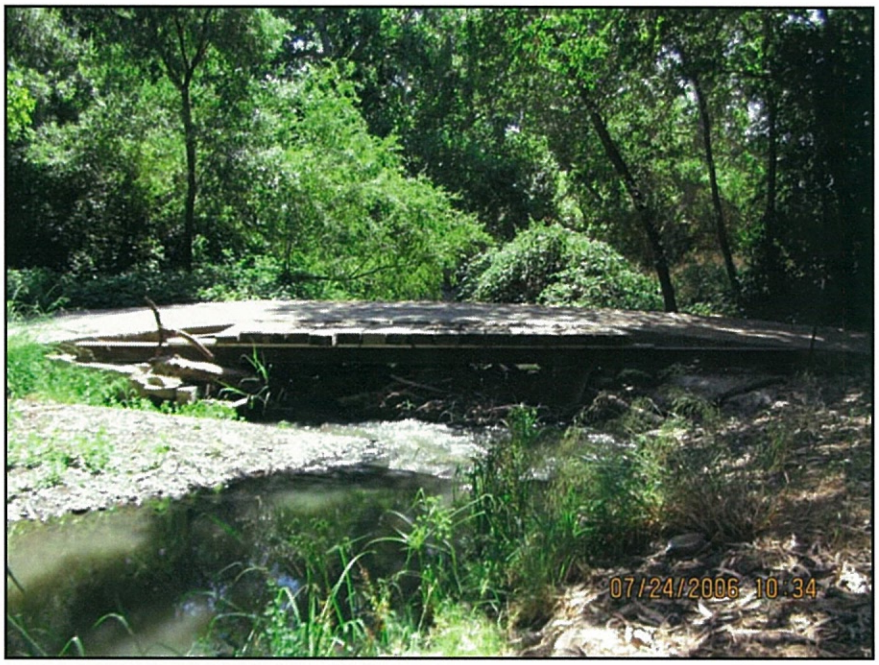


Photo 48. Road Crossing on Llagas Creek below Chesbro Reservoir View downstream. Bed material is composed of gravel. Note debris from recent flood at bridge.

Photo 49. Llagas Creek Upstream of Watsonville Road
View upstream. Note heavily vegetated channel.

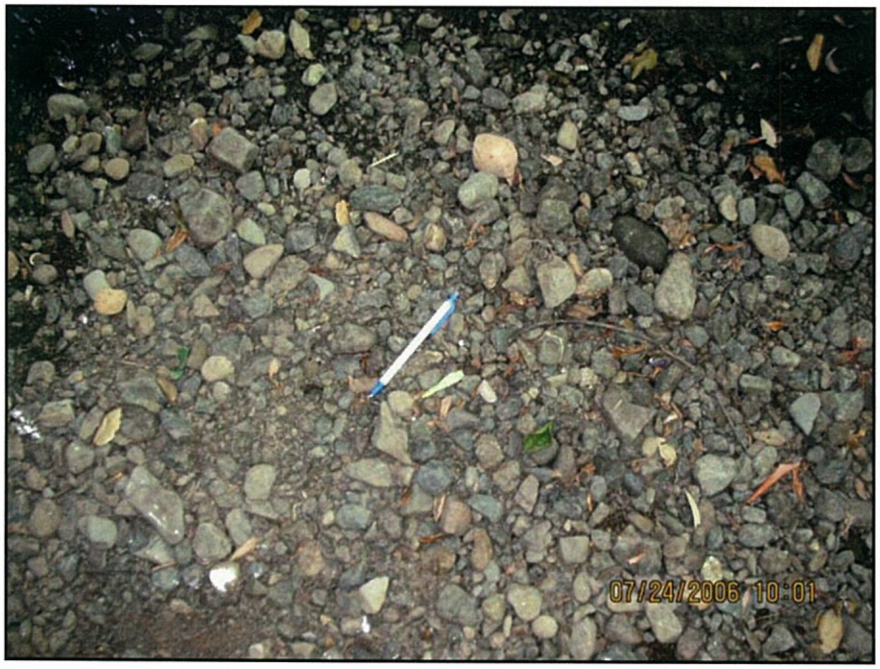


Photo 50. Bed Material in Llagas Creek at Santa Teresa Boulevard

Photo 51. Lake Silveria East of Llagas Creek Upstream of Monterey Road

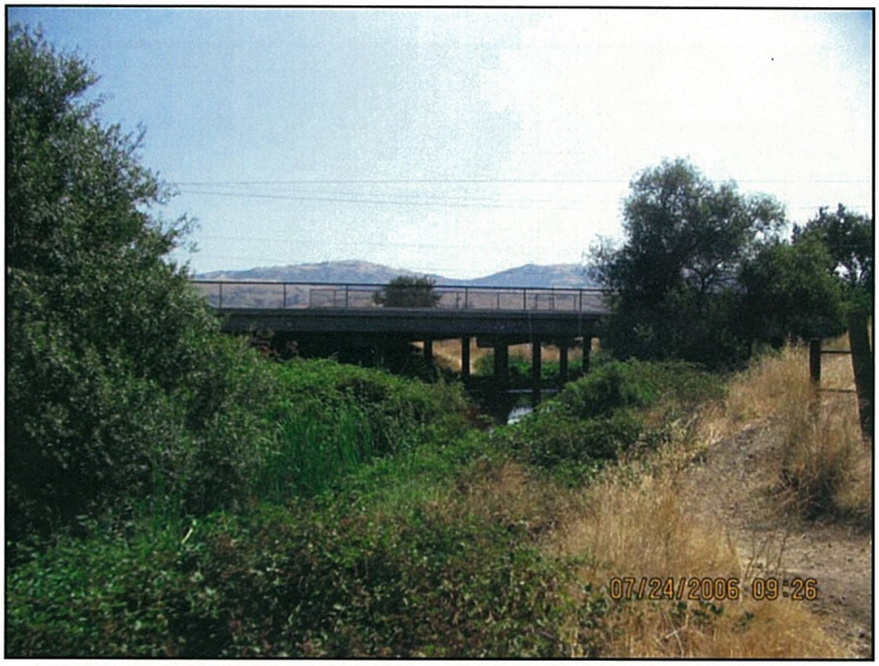


Photo 52. Llagas Creek at Monterey Road
View downstream. Note heavy vegetation along creek banks.

Photo 53. Llagas Creek above Llagas Avenue Bridge
View upstream. Note heavily vegetated channel.

Photo 54. Llagas Creek below Llagas Avenue Bridge
View downstream. Note heavy in-channel vegetation.

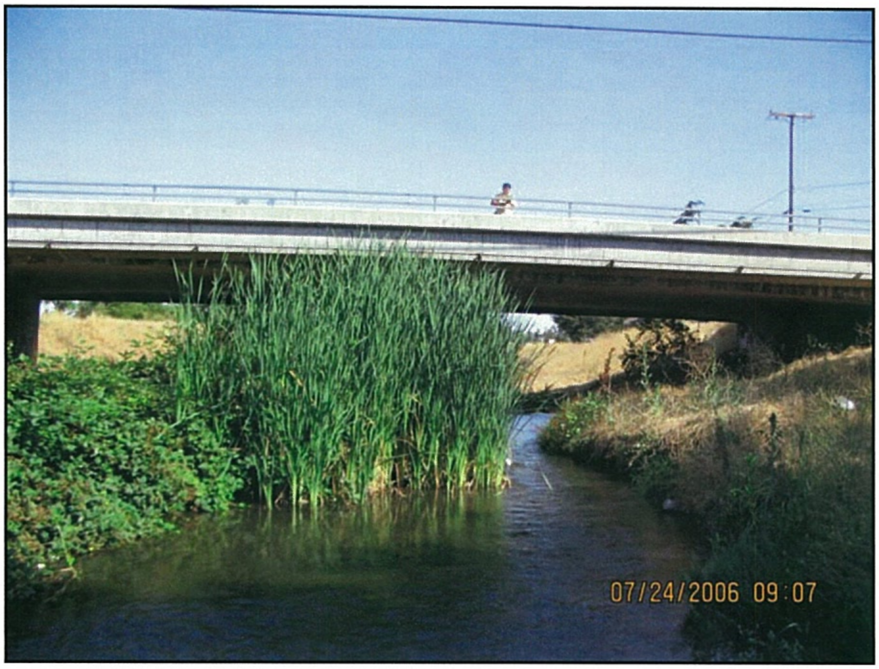


Photo 55. Llagas Creek at East San Martin Avenue Bridge
View upstream. Note dense in-channel vegetation. Flow velocity is about 1 ft/s.

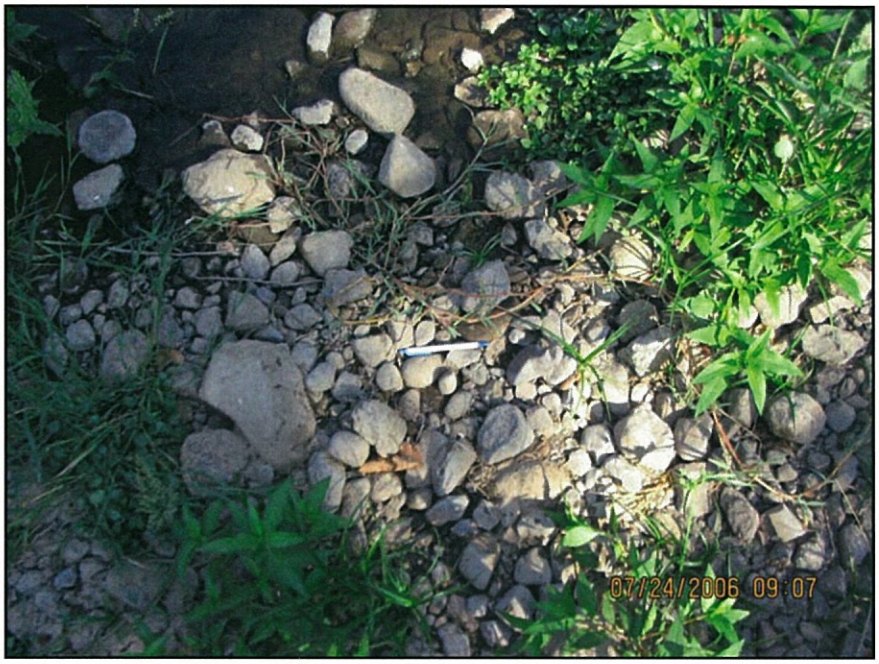


Photo 56. Bed in Llagas Creek Immediately Downstream of East San Martin Avenue
The bed is composed of medium to coarse gravel.

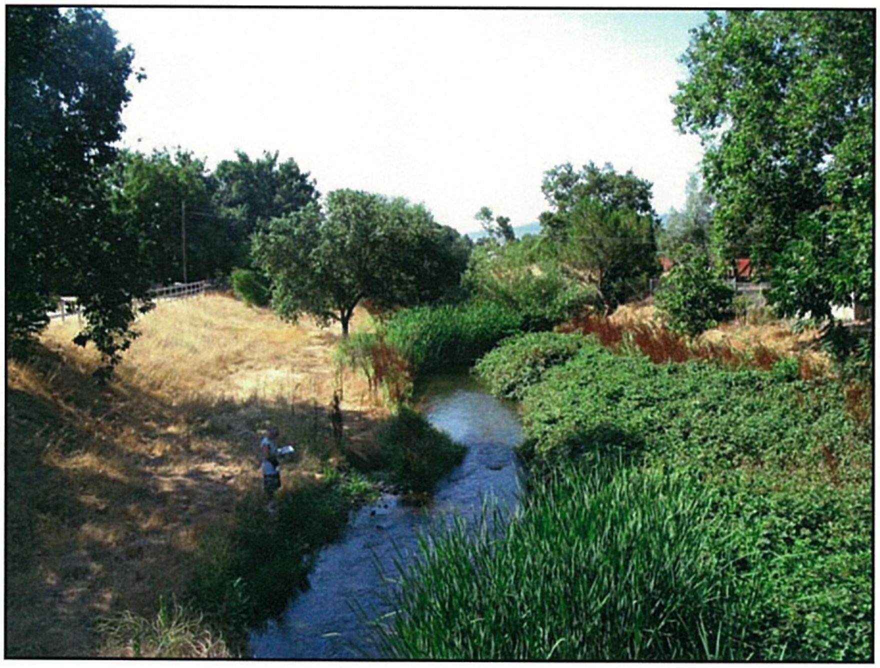


Photo 57. Llagas Creek below East San Martin Avenue
View downstream.

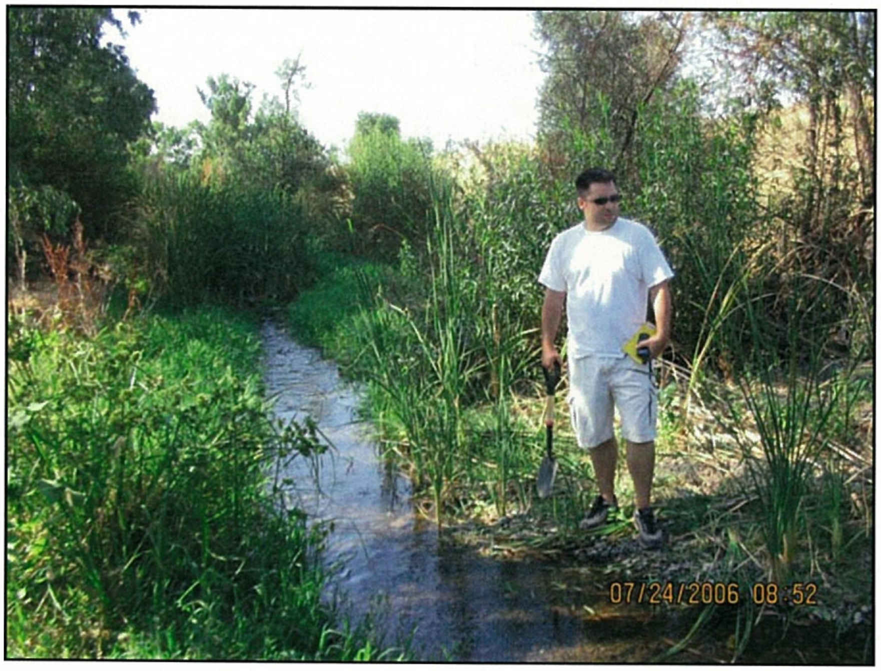


Photo 58. Llagas Creek below Church Avenue
View downstream. Note heavily vegetated channel.

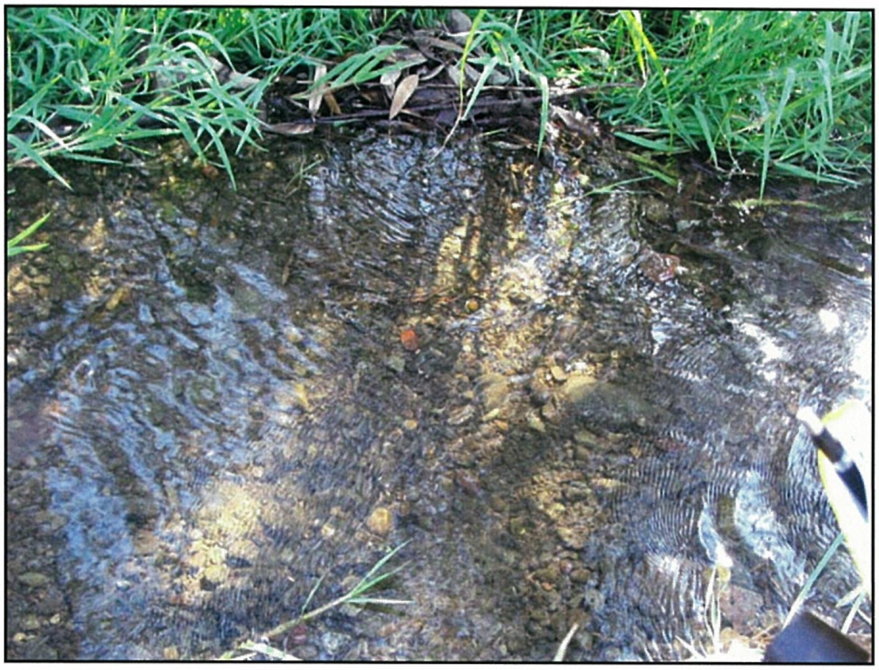


Photo 59. Bed Material in Llagas Creek below Church Avenue
The material is composed of fine gravel.

Photo 60. Lake at Llagas Creek Upstream of Church Avenue

Photo 61. Levee Between Lake and Llagas Creek Downstream of Church Avenue
Creek is on left side of photo. View downstream.

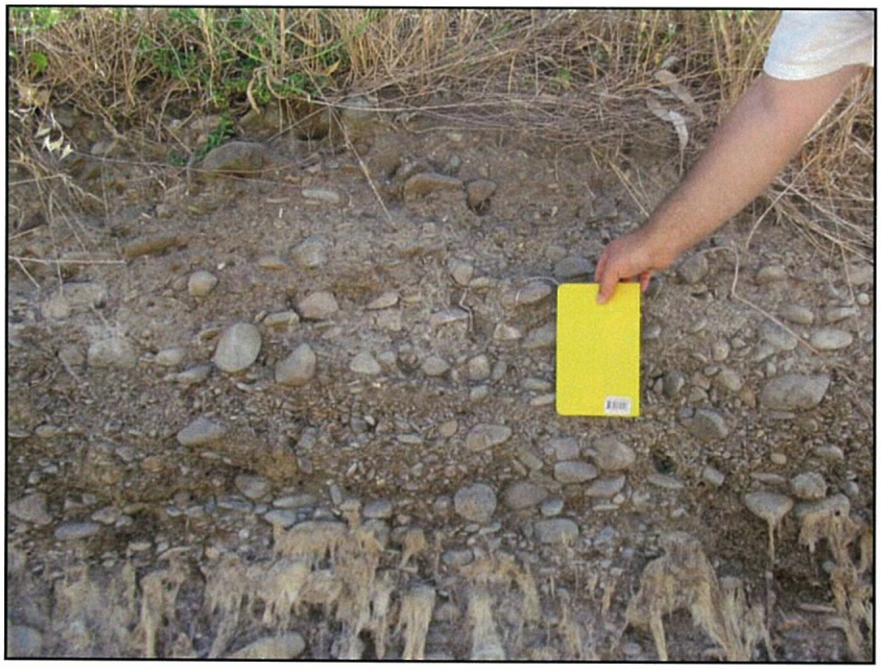


Photo 62. Bank of Llagas Creek Upstream of Masten Avenue
The bank is composed of gravel cemented with clay and silt.

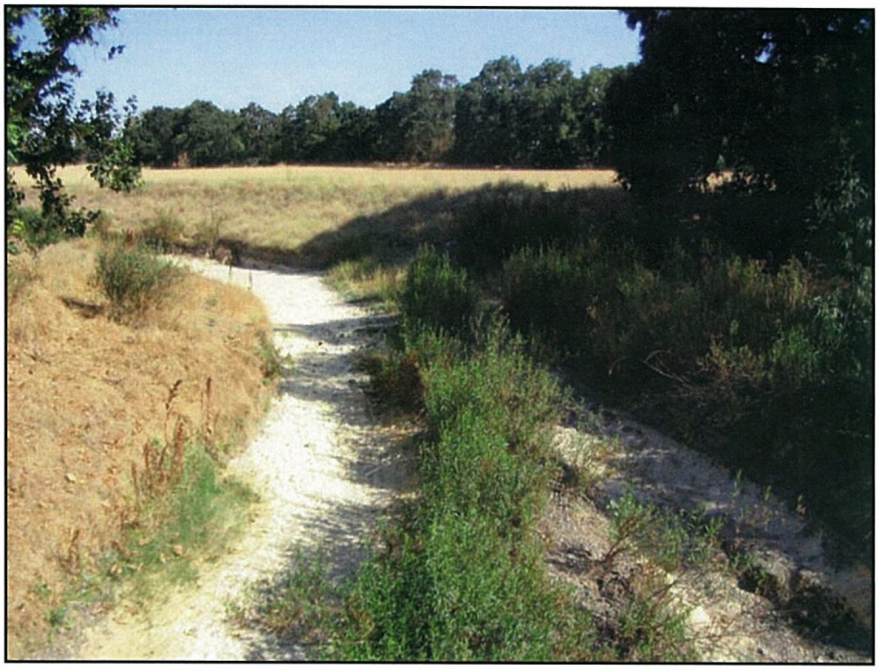


Photo 63. Llagas Creek Immediately Upstream of Masten Avenue Bridge
View upstream. In-channel vegetation traps sediment and promotes bar development.

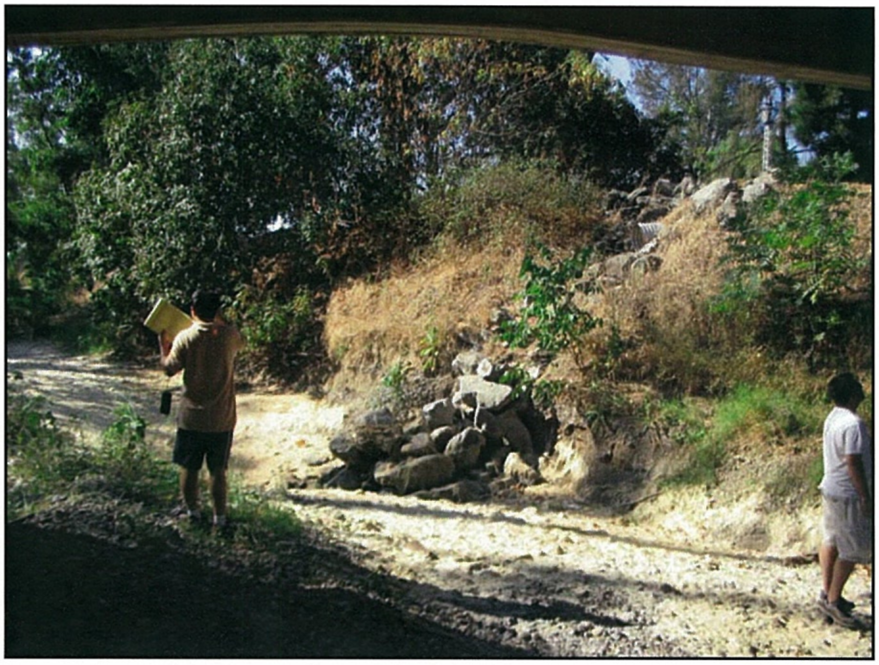


Photo 64. Rocks Placement at Small Culvert Outlet Immediately Downstream of Masten Avenue
View downstream. Note bed scour around rocks.

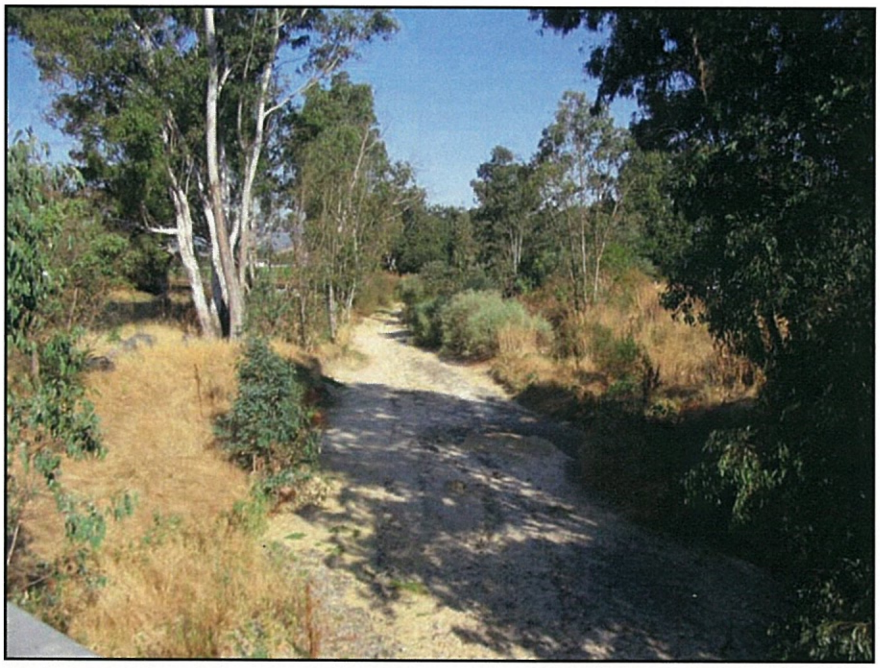


Photo 65. Llagas Creek above Rucker Avenue
View upstream.

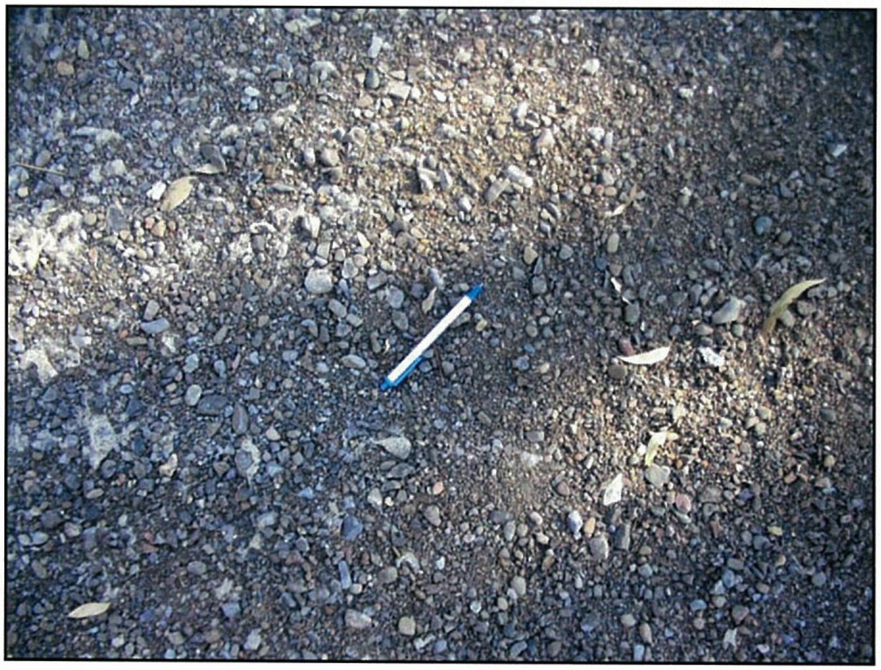


Photo 66. Fine Gravel About 100 Feet Upstream of Rucker Avenue
Stream bed in this reach is flat, without any bed forms.

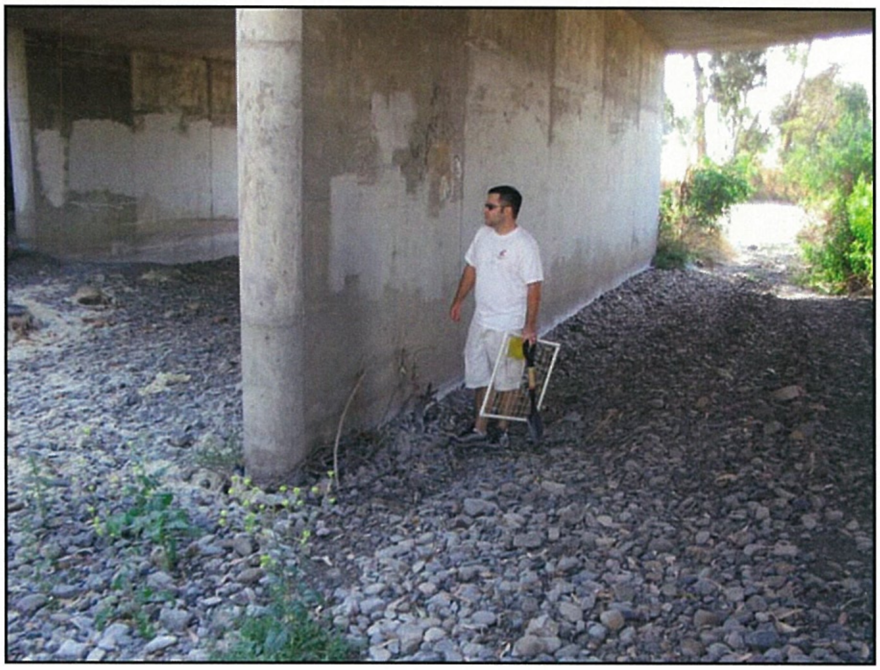


Photo 67. Bed Scour at Pier of Rucker Avenue Bridge

Photo 68. Llagas Creek below Rucker Avenue
View downstream.

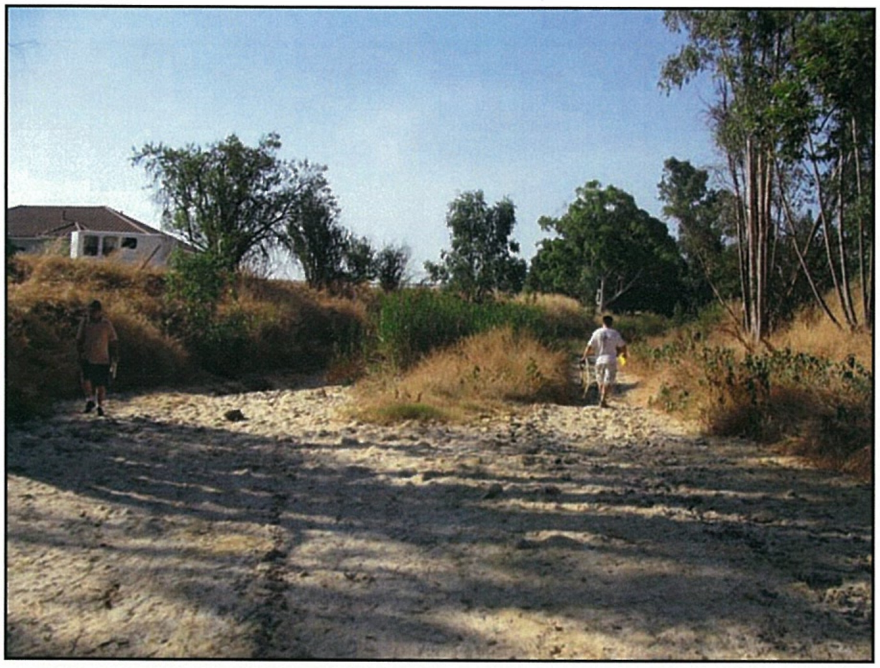


Photo 69. Channel Bifurcation in Llagas Creek
It is located around vegetated gravel bar approximately 300 ft downstream of Rucker Avenue. View downstream.

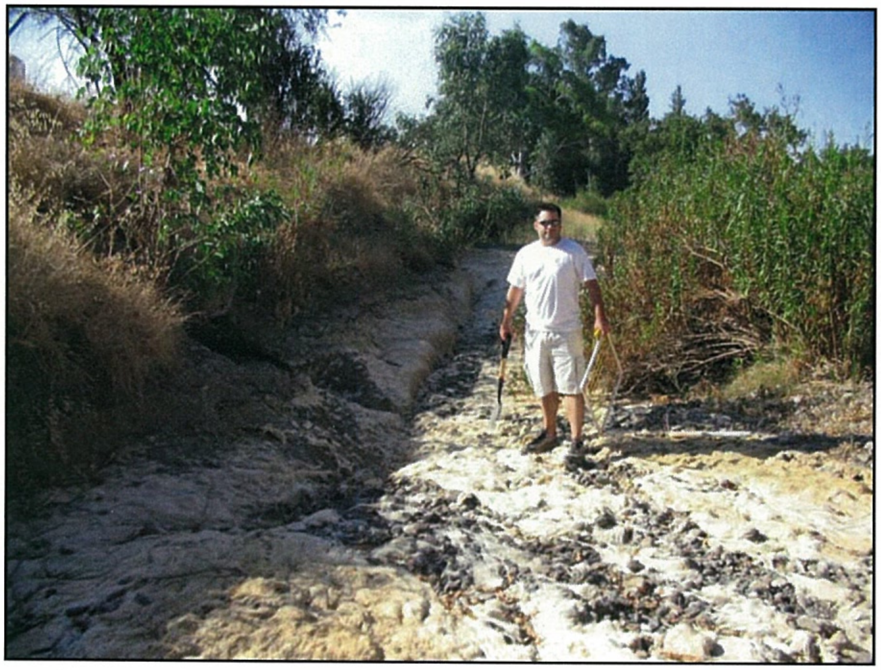


Photo 70. Bed Scour Between Min-channel Gravel Bar and East Bank of Llagas Creek
It is located approximately 300 ft below Rucker Avenue. View downstream.

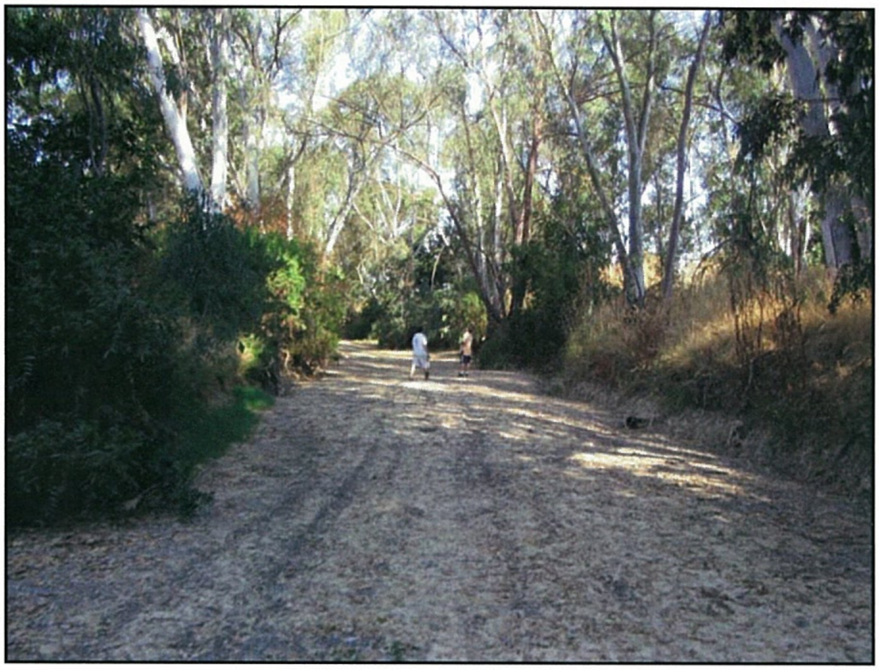


Photo 71. Llagas Creek Upstream of Buena Vista Avenue
View upstream. Note mature trees along creek banks. Bed is composed of loose gravel. Note absence of any noticeable bed forms.

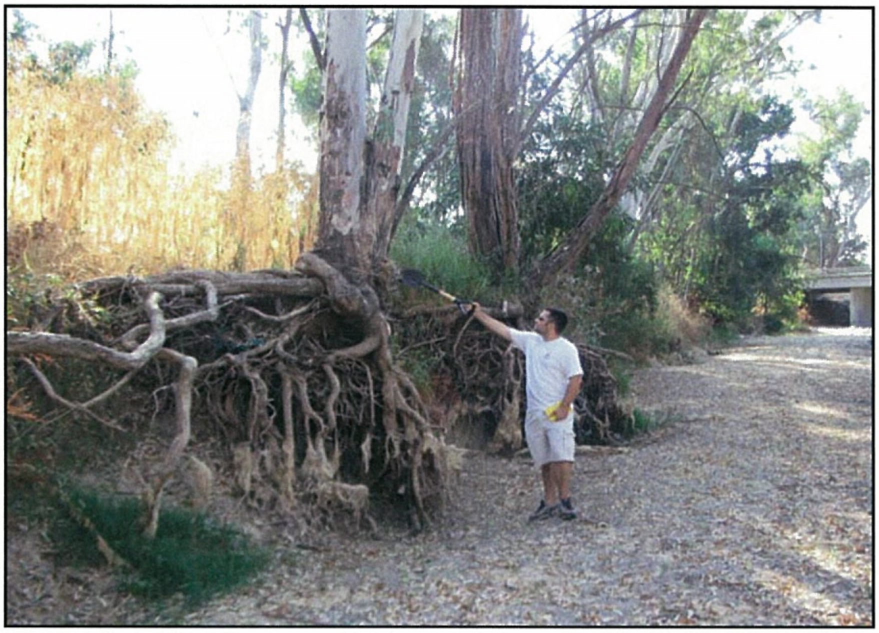


Photo 72. Exposed Tree Roots in Llagas Creek Upstream of Buena Vista Avenue
View downstream. Person on photo points at high water mark (debris) located about 5 ft above bed.

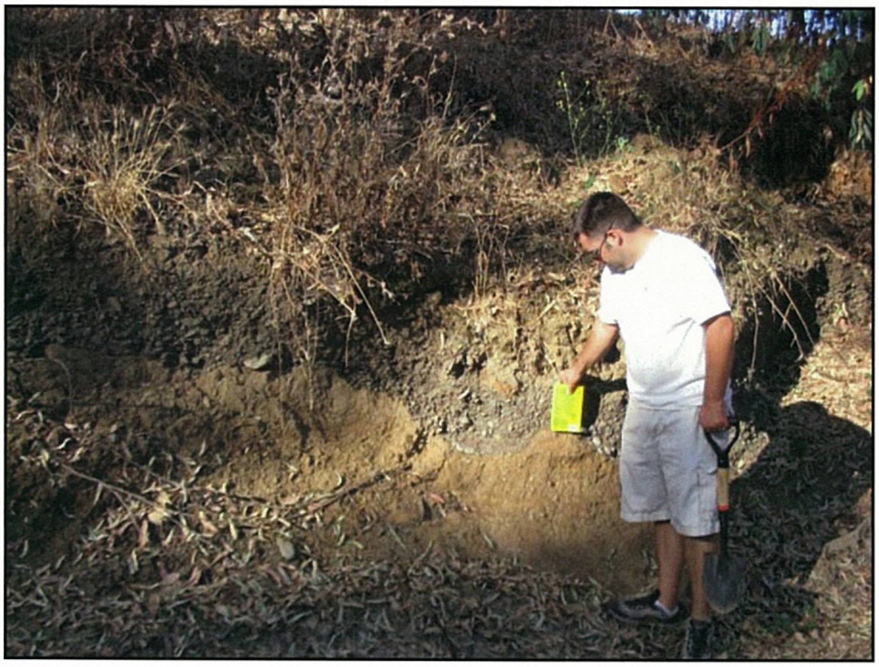


Photo 73. Bank of Llagas Creek Upstream of Buena Vista Avenue
The bank is composed of sand with layers of 2-5 mm gravel.

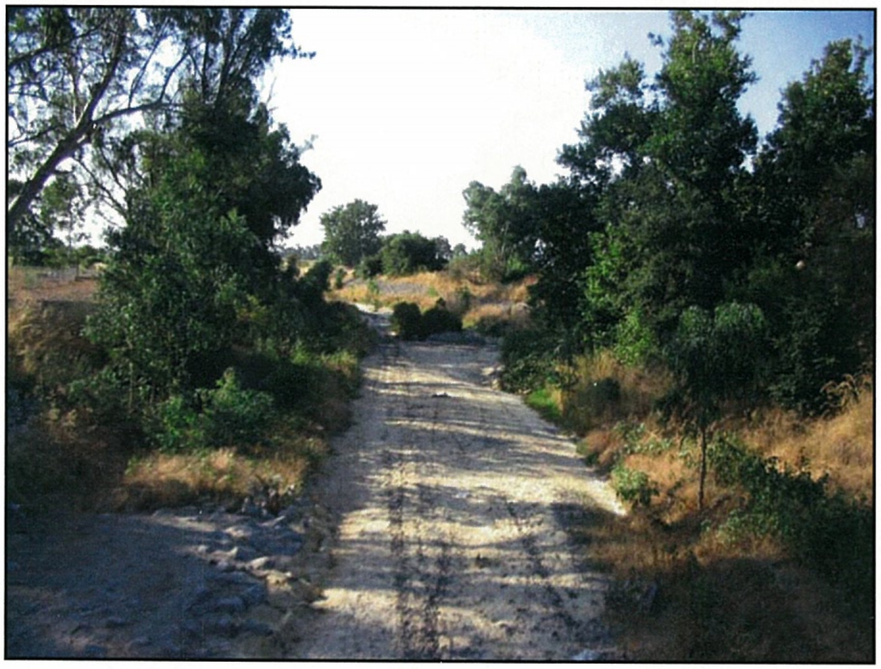


Photo 74. Llagas Creek Downstream of Buena Vista Avenue
View downstream.

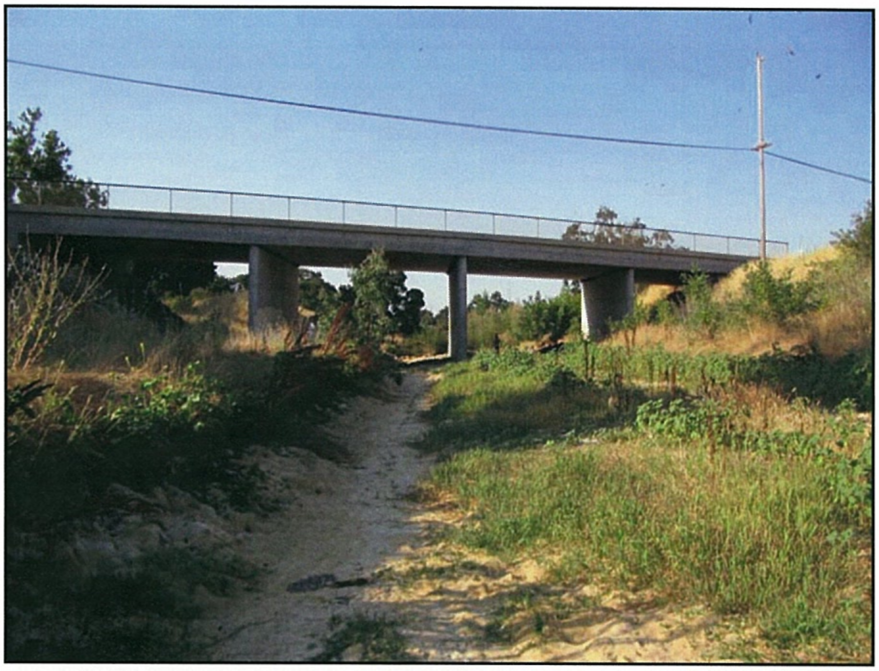


Photo 75. Llagas Creek at Leavesley Road
View downstream.

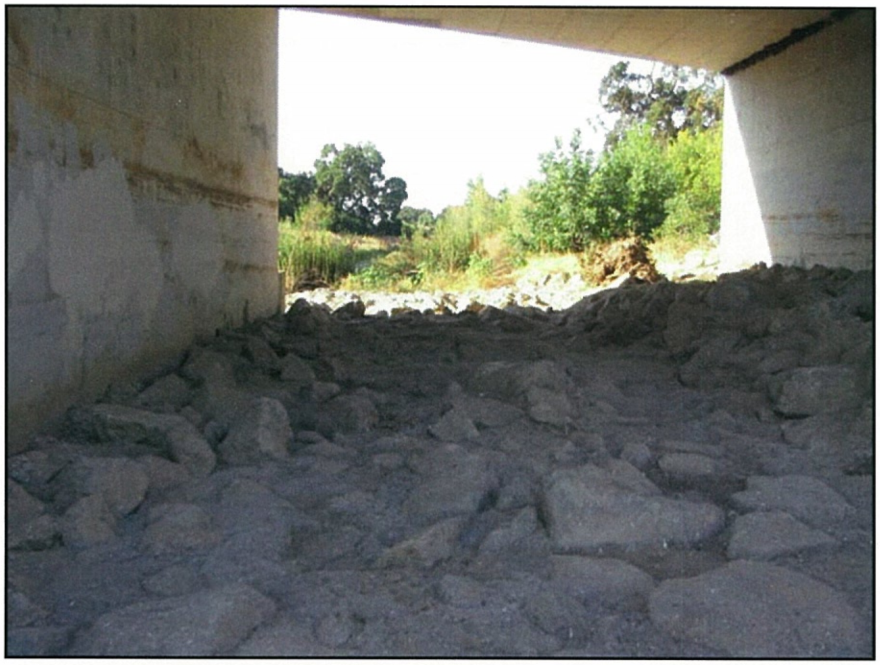


Photo 76. Grouted Rocks under Leavesley Road Bridge

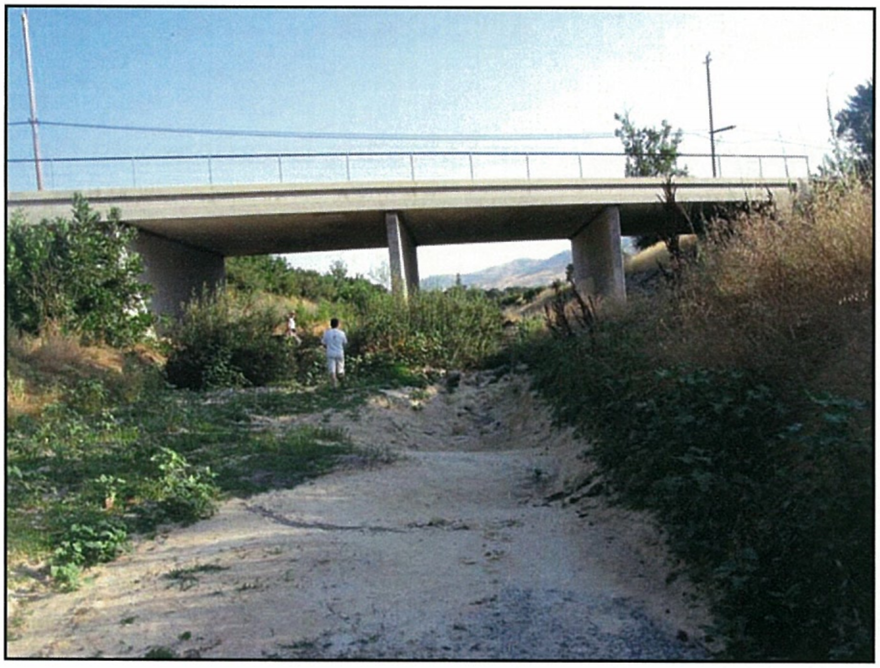


Photo 77. Llagas Creek below Leavesley Road Bridge.
View upstream. Note scour hole immediately downstream of bridge.

APPENDIX B

Site Photographs at Sediment Sampling Locations and Grain Size Distributions

Photo B-1. Bed Material Sampling Location 1
(West Little Llagas Creek Downstream of Private Road).

Photo B-2. Bed Material Sampling Location 2
(West Little Llagas Creek at Hillwood Lane)

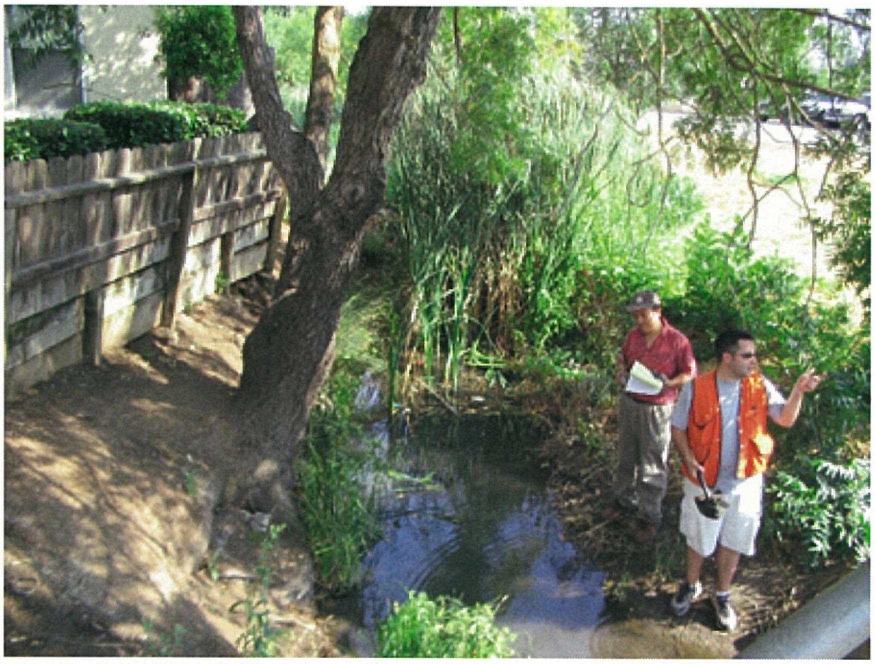


Photo B-3. Bed Material Sampling Location 3
(West Little Llagas Creek Downstream of West Main Ave.).

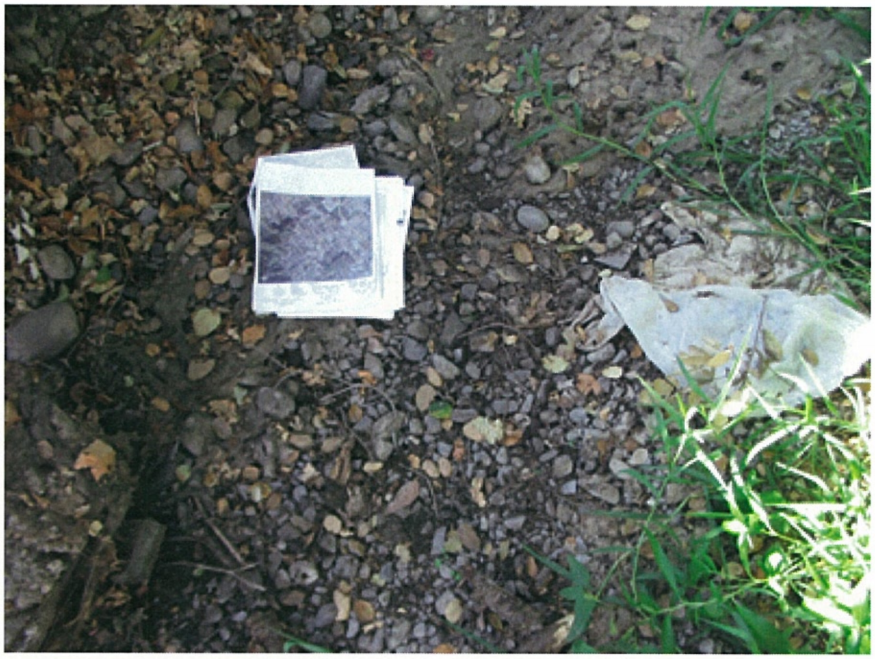


Photo B-4. Bed Material Sampling Location 4
(West Little Llagas Creek Upstream of West 5th Street).

Photo B-5. Bed Material Sampling Location 5
(West Little Llagas Creek Downstream of Spring Ave.).

Photo B-6. Bed Material Sampling Location 6
(West Little Llagas Creek Upstream of South La Crosse)

Photo B-7. Bed Material Sampling Location 7
(East Little Llagas Creek Downstream of Watsonville Road)

Photo B-8. Bed Material Sampling Location 8
(East Little Llagas Creek Downstream of East San Martin Ave.).

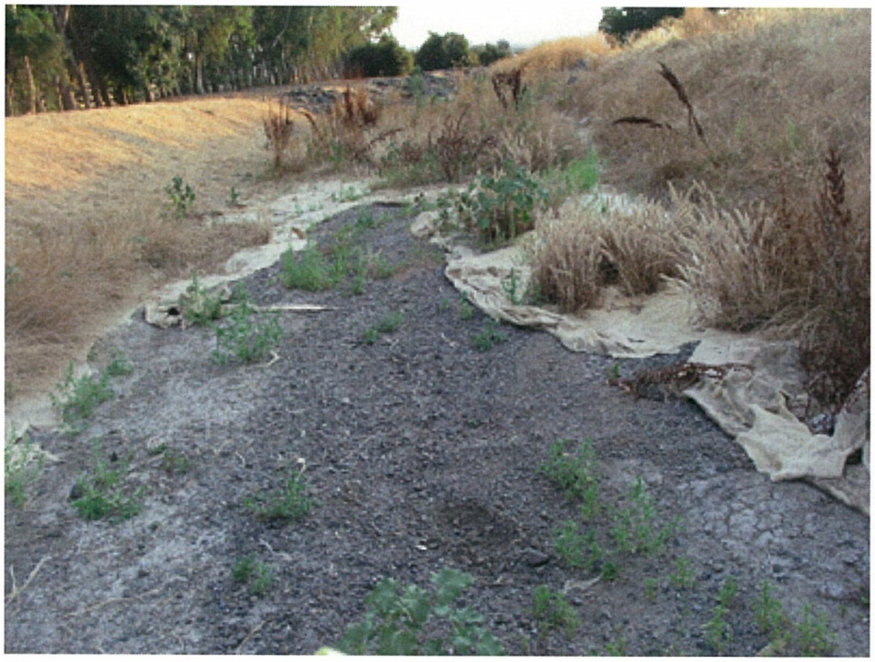


Photo B-9. Bed Material Sampling Location 9
(East Little Llagas Creek Upstream of Sycamore Ave.).

Photo B-10. Bed Material Sampling Location 10
(East Little Llagas Creek Downstream of Church Avenue).

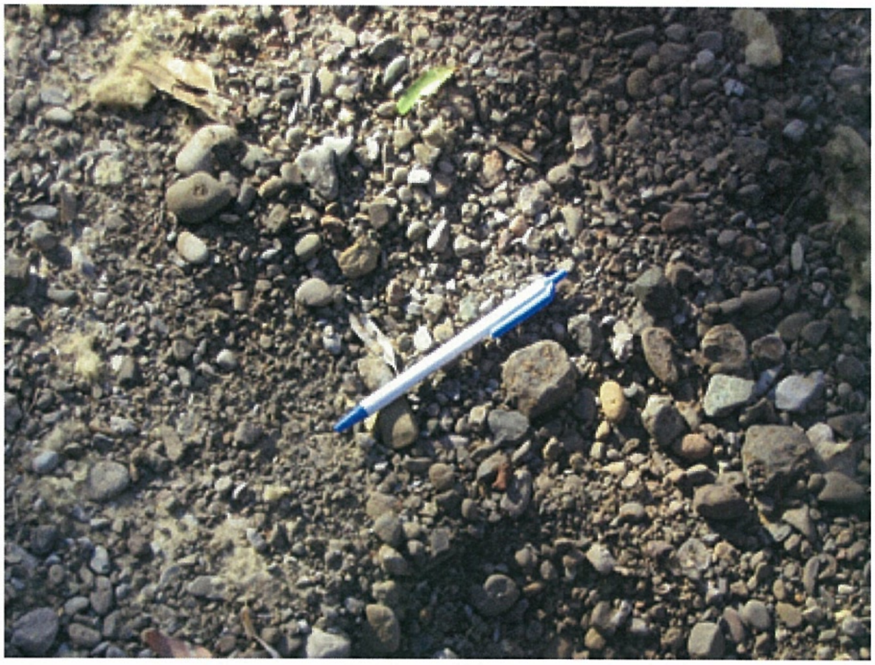


Photo B-11. Bed Material Sampling Location 11 (Llagas Creek Upstream of Buena Vista Ave.).

Photo B-12. Bed Material Sampling Location 12 (Llagas Creek Upstream of Rucker Ave.).

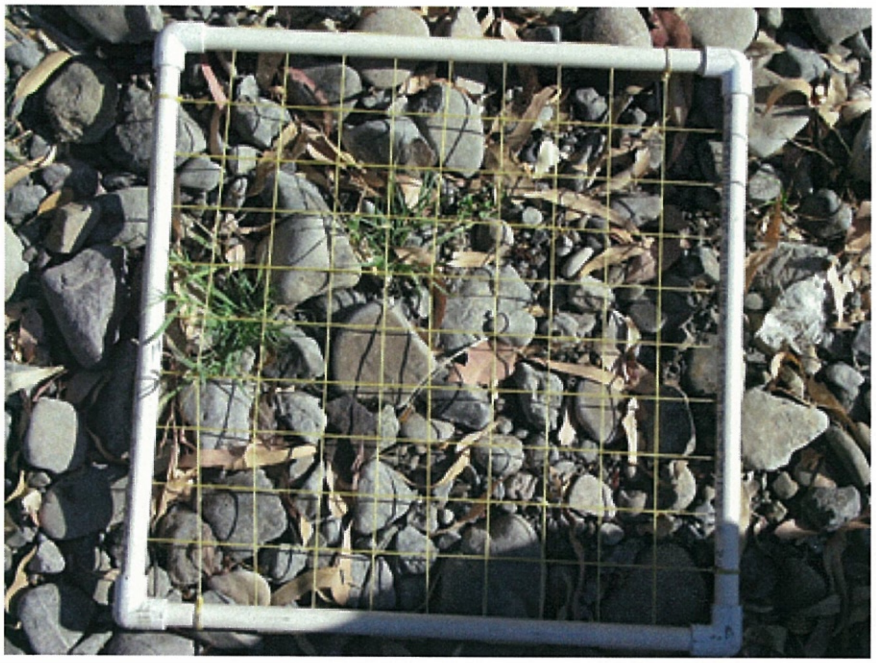


Photo B-13. Bed Material surface Sampling Location 12a (Llagas Creek Next to Rucker Ave.).

Photo B-14. Bed Material Sampling Location 13 (Llagas Creek Downstream of Church Ave.).

Photo B-15. Bed Material Sampling Location 14
(Llagas Creek Upstream of Monterey Road).

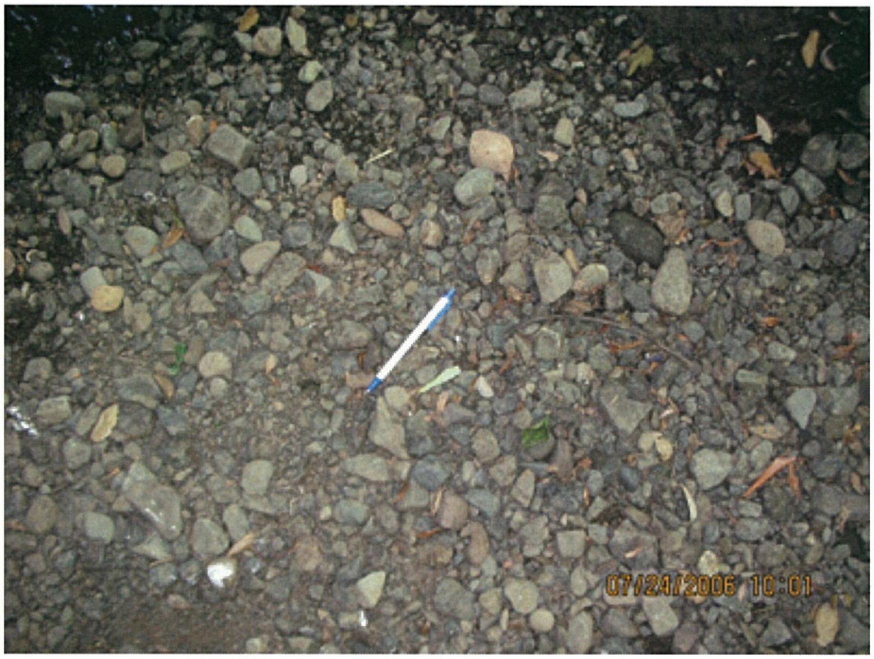
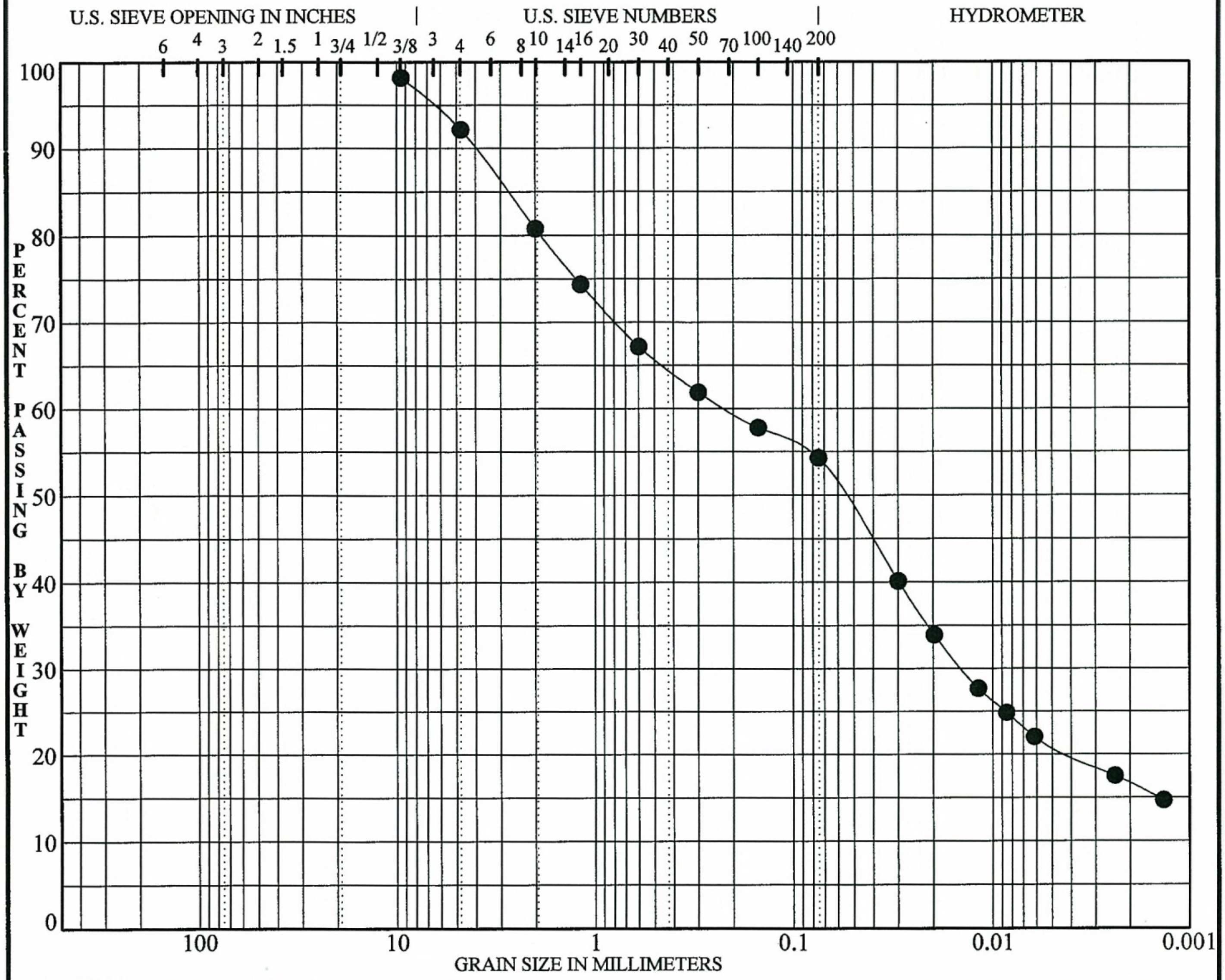


Photo B-16. Bed Material Sampling Location 15
(Llagas Creek at Santa Teresa Blvd.).



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	01	0.5	54	92	Brown sandy SILT	ML

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GRADATION TEST DATA

LLAGAS CREEK
Llagas Creek, California

FIGURE
1

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590-16

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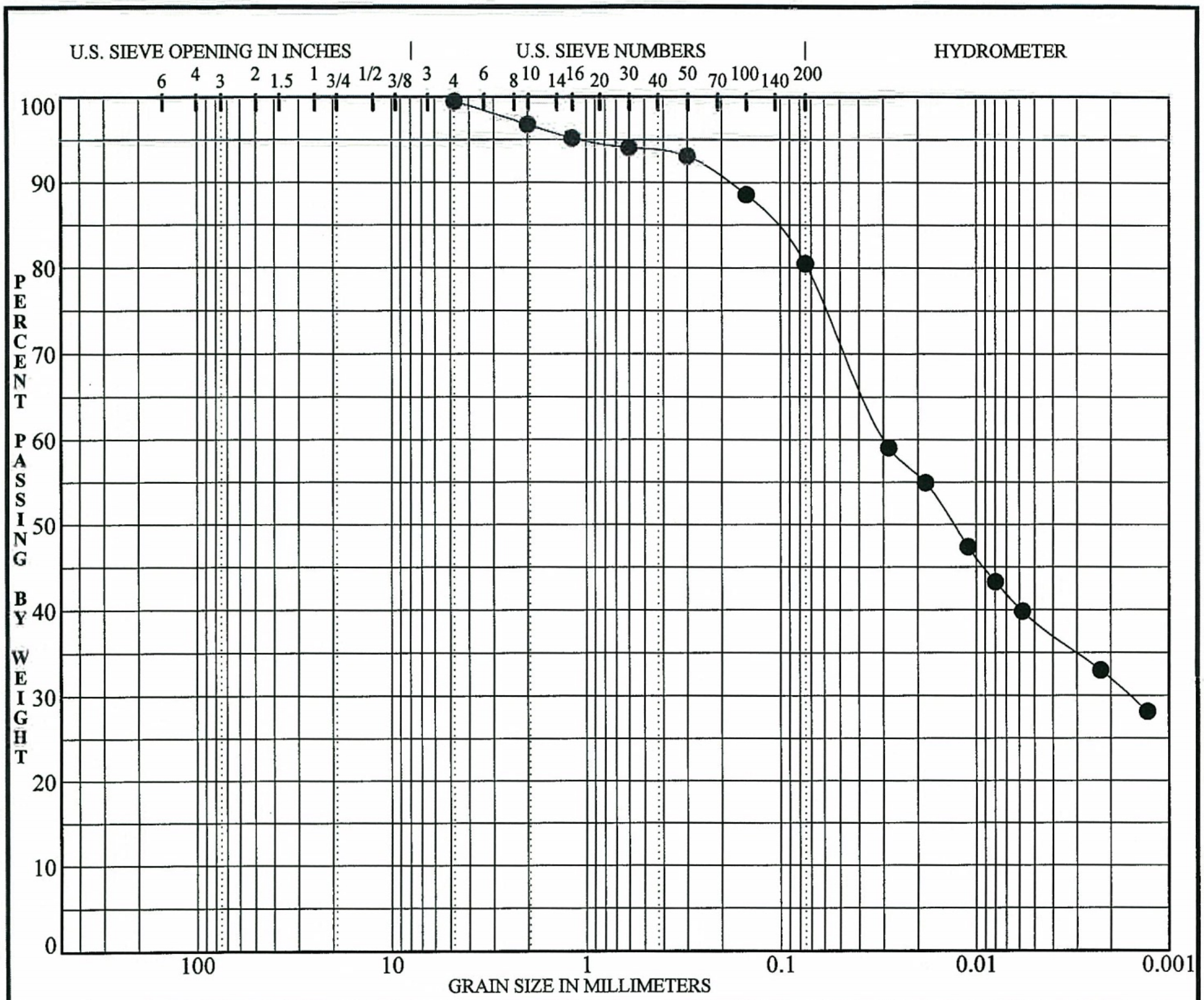

Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	02	0.5	81	100	Brown sandy SILT	ML

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LLAGAS CREEK

Llagas Creek, California

FIGURE

2

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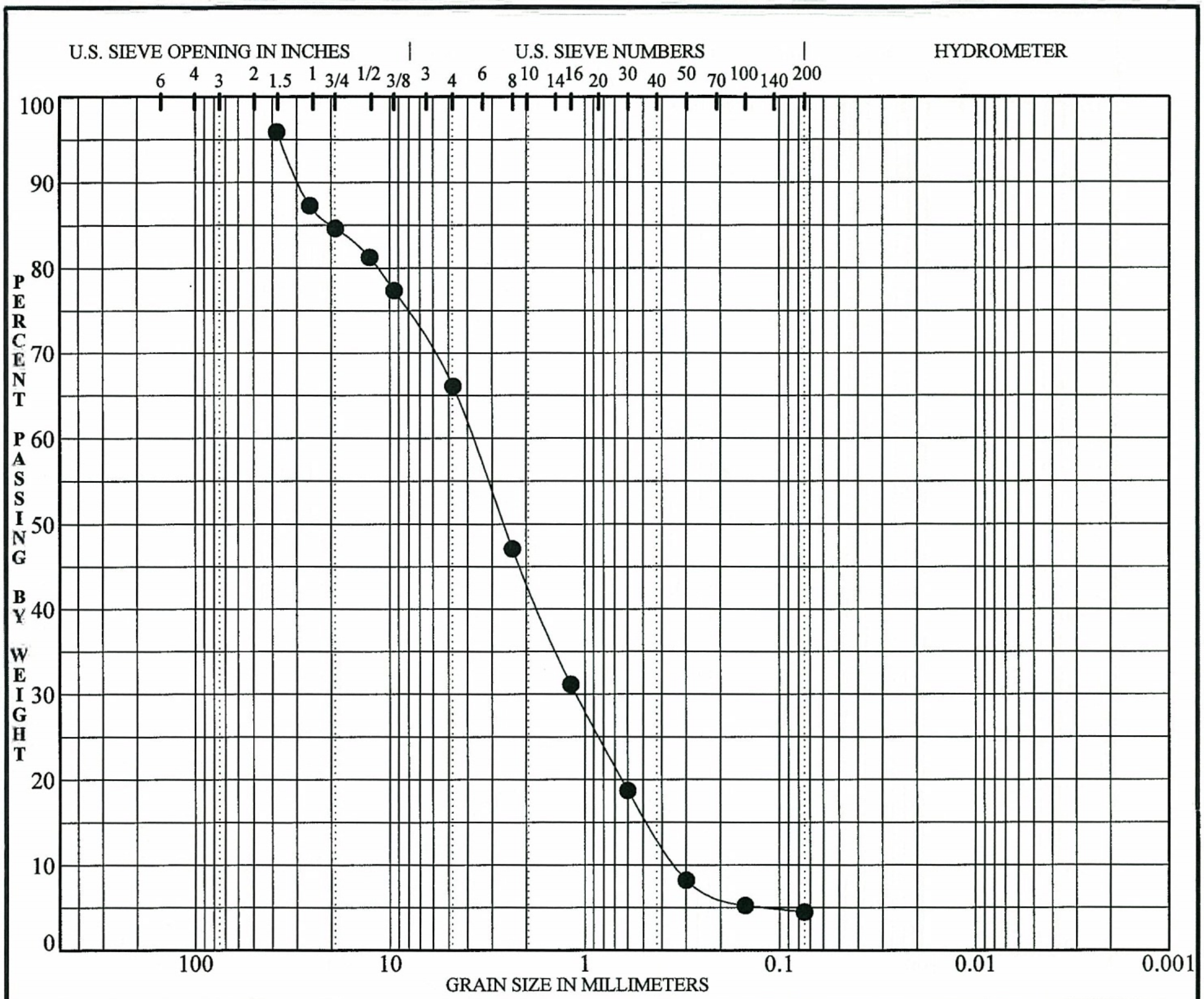
590-16



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	03	0.5	5	66	Gray well-graded SAND with gravel	SW

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LLAGAS CREEK

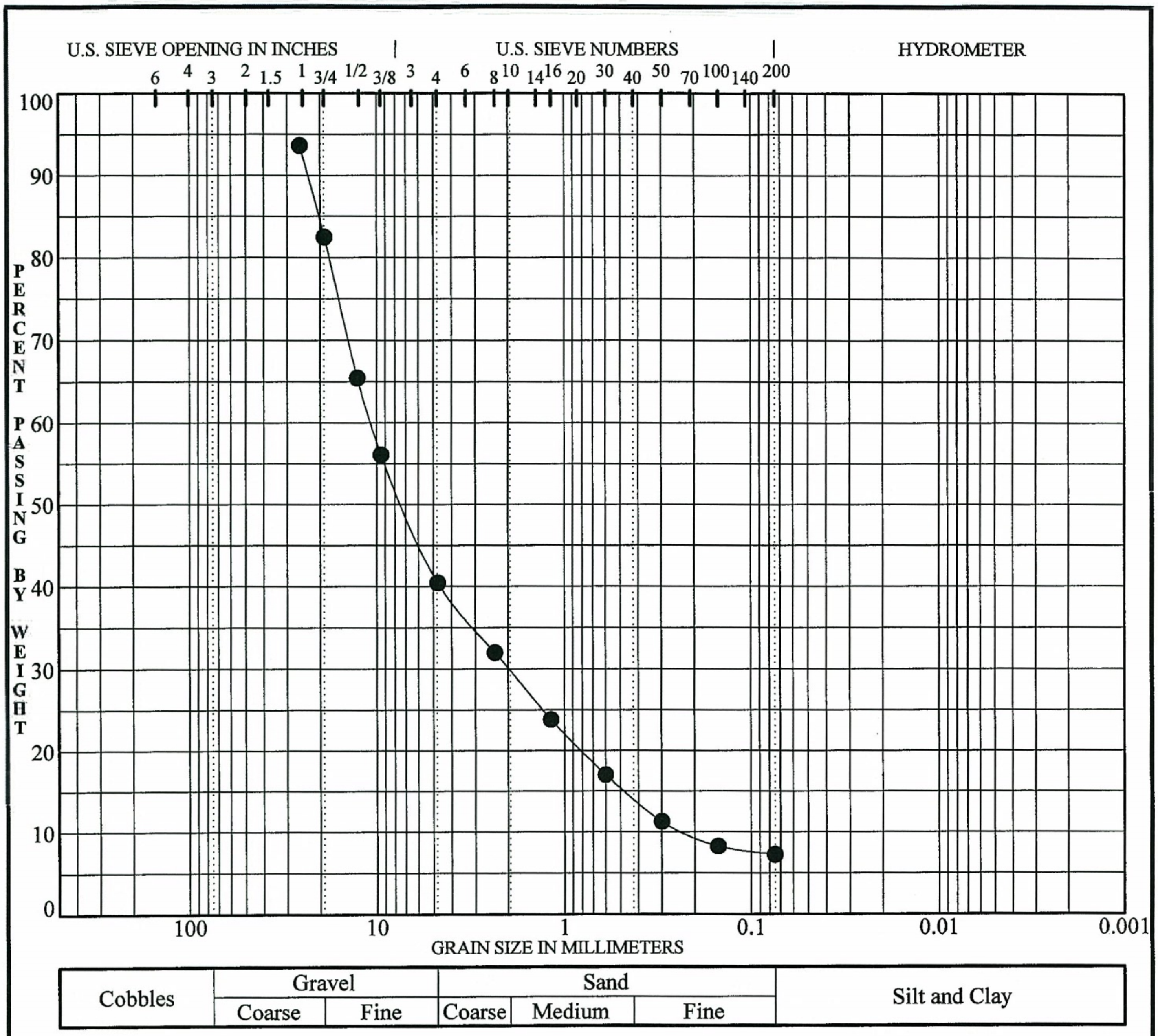
Llagas Creek, California

FIGURE

3

PROJECT No.

590-16



Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	04	0.5	7	40	Gray well-graded GRAVEL with silt and sand	GW-GM

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LLAGAS CREEK

Llagas Creek, California

FIGURE

4

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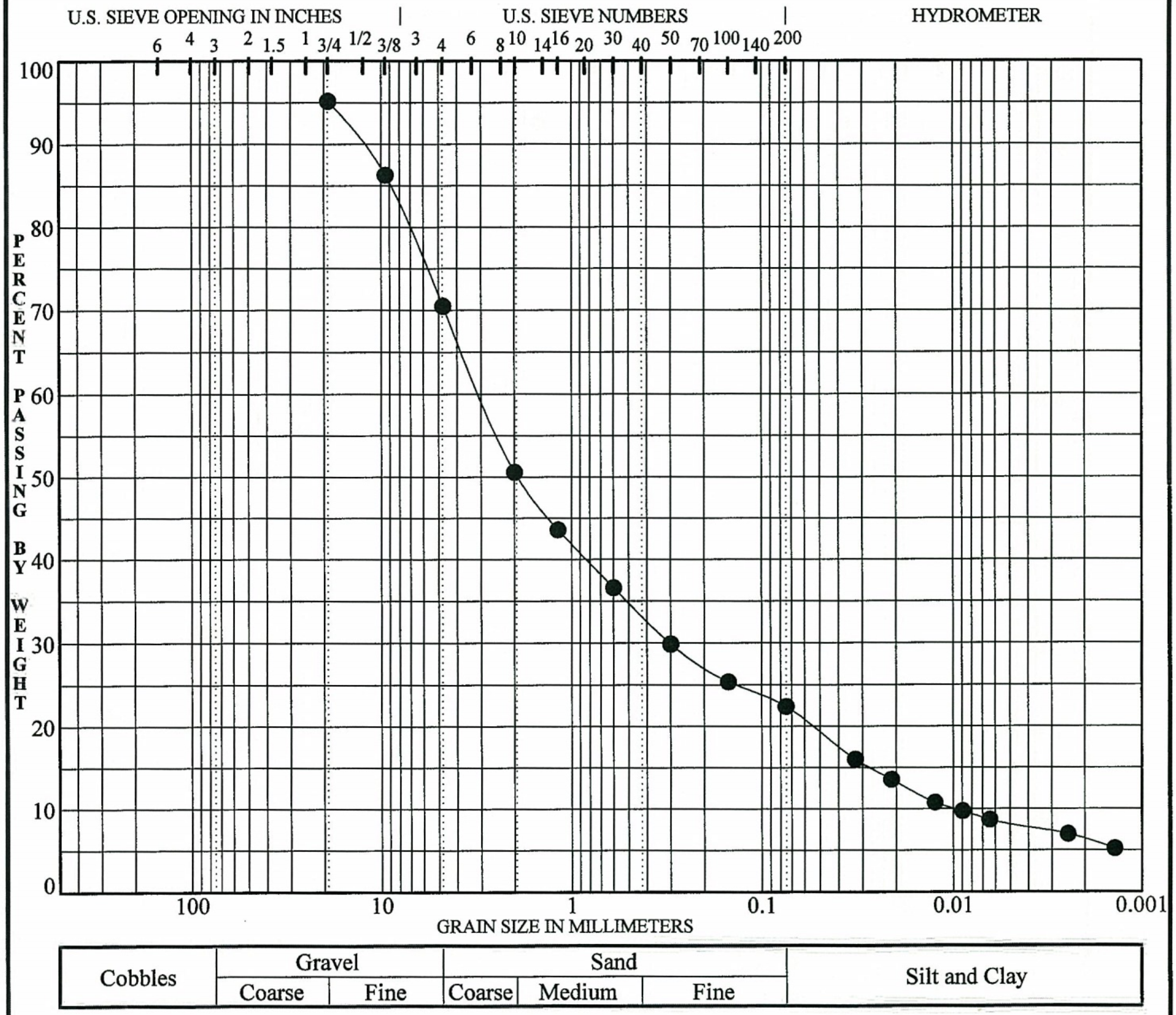
590-16



Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	05	0.5	22	71	Light brown silty SAND with gravel	SM

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LLAGAS CREEK

Llagas Creek, California

FIGURE

5

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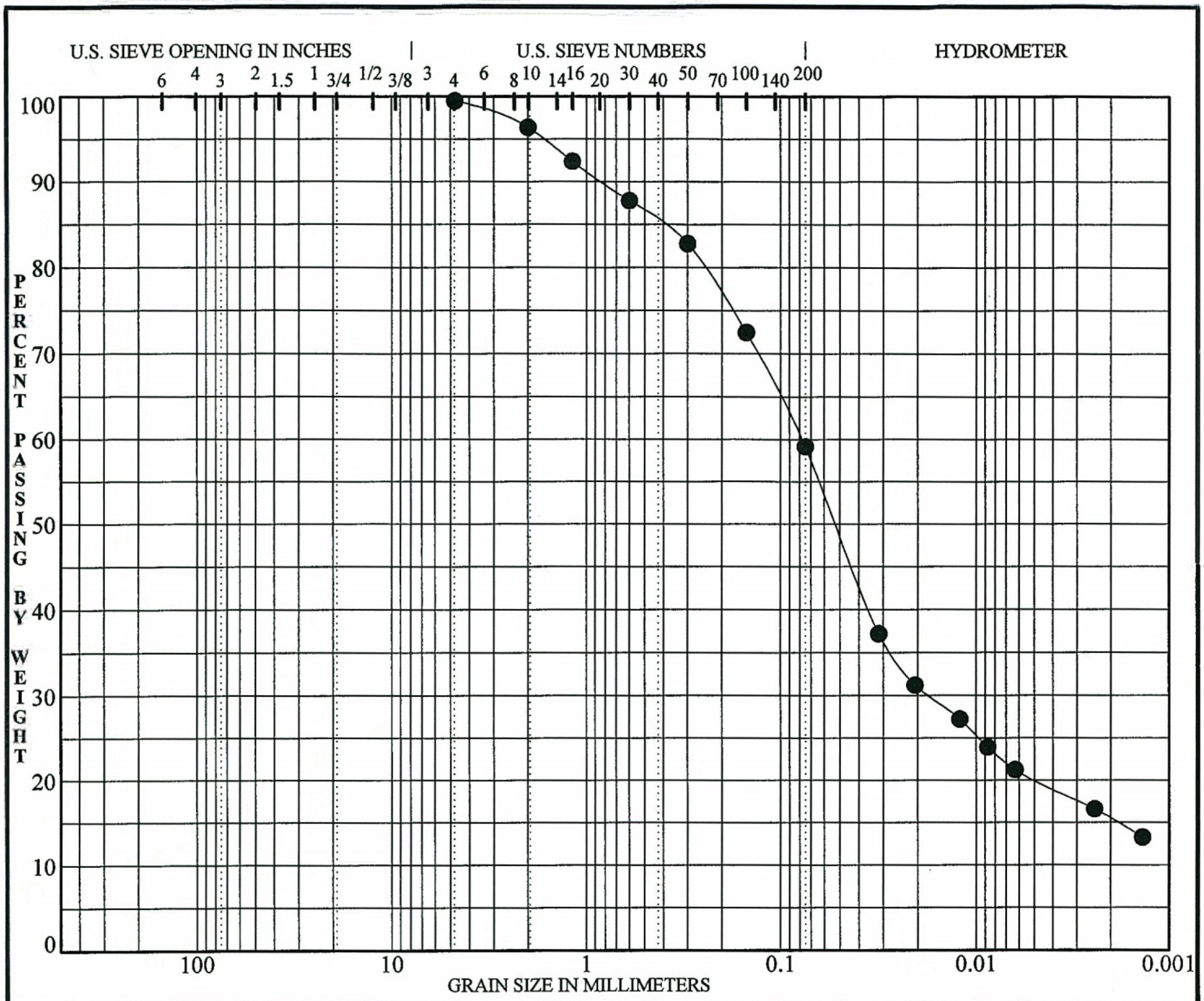
590-16



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	06	0.5	59	100	Brown sandy SILT	ML

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LLAGAS CREEK

Llagas Creek, California

FIGURE

6

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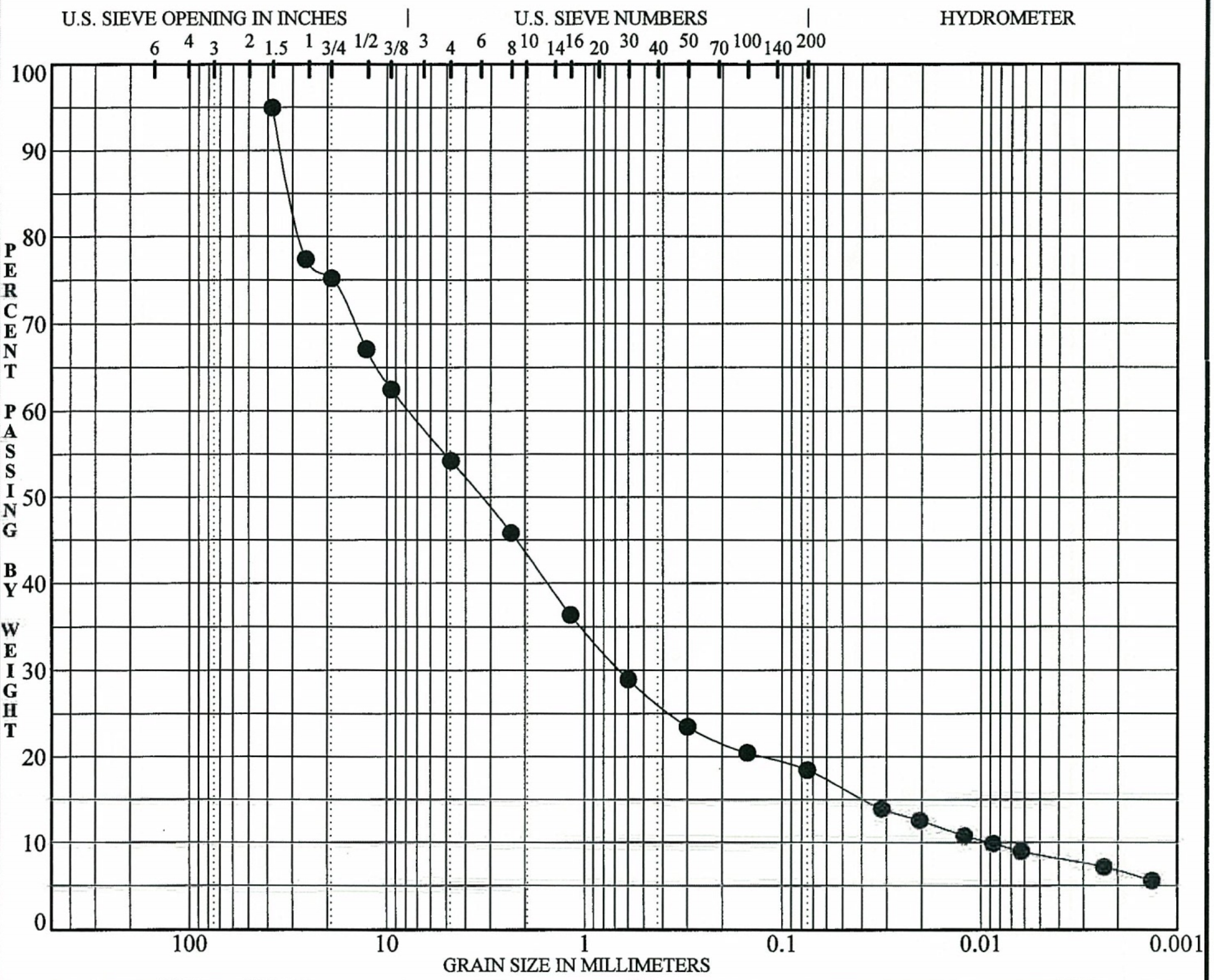

Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	07	0.5	18	54	Gray brown silty SAND with gravel	SM

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LLAGAS CREEK
Llagas Creek, California

FIGURE
7

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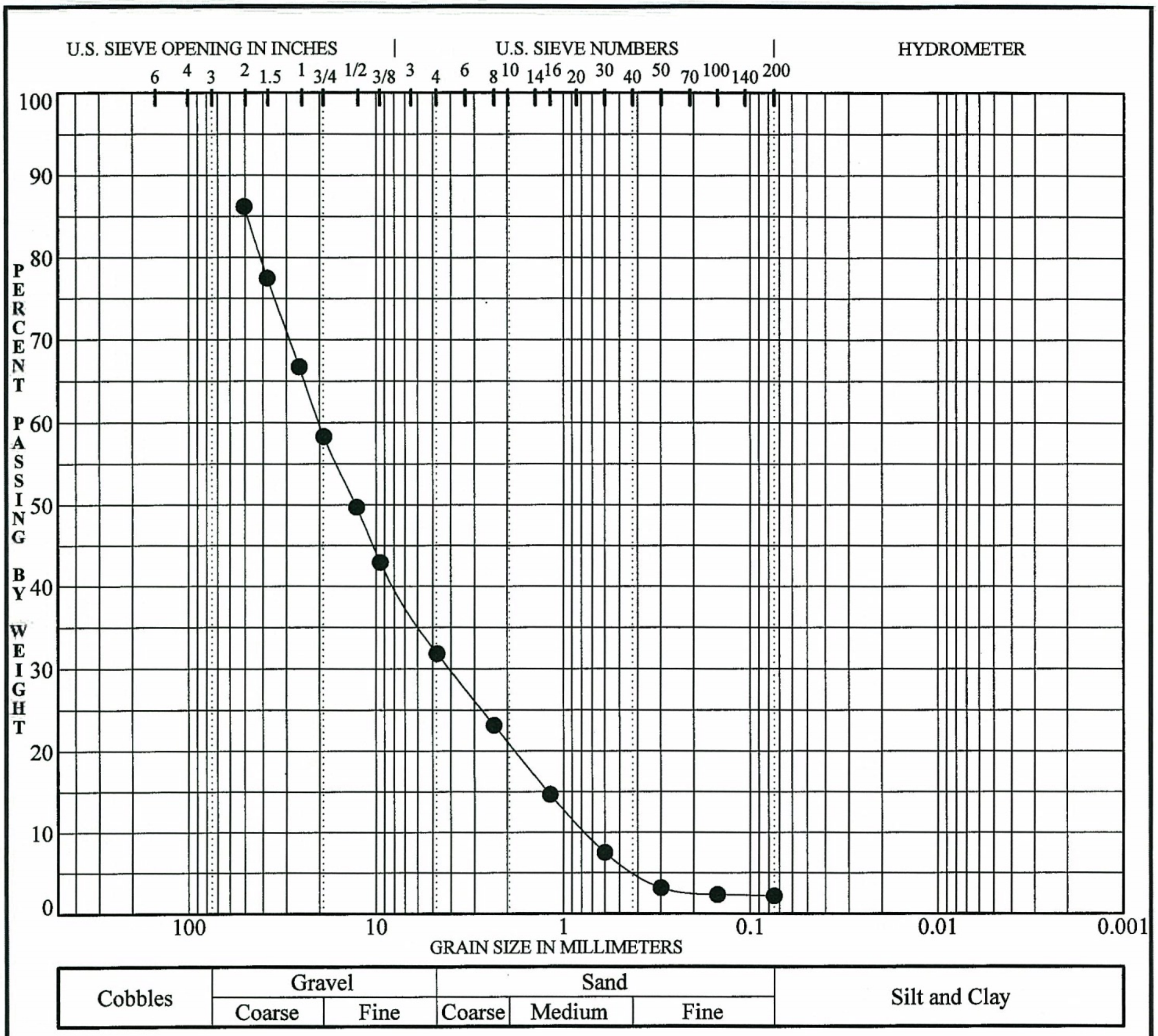
590-16



Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	08	0.5	2	32	Gray well-graded GRAVEL with sand	GW

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LLAGAS CREEK

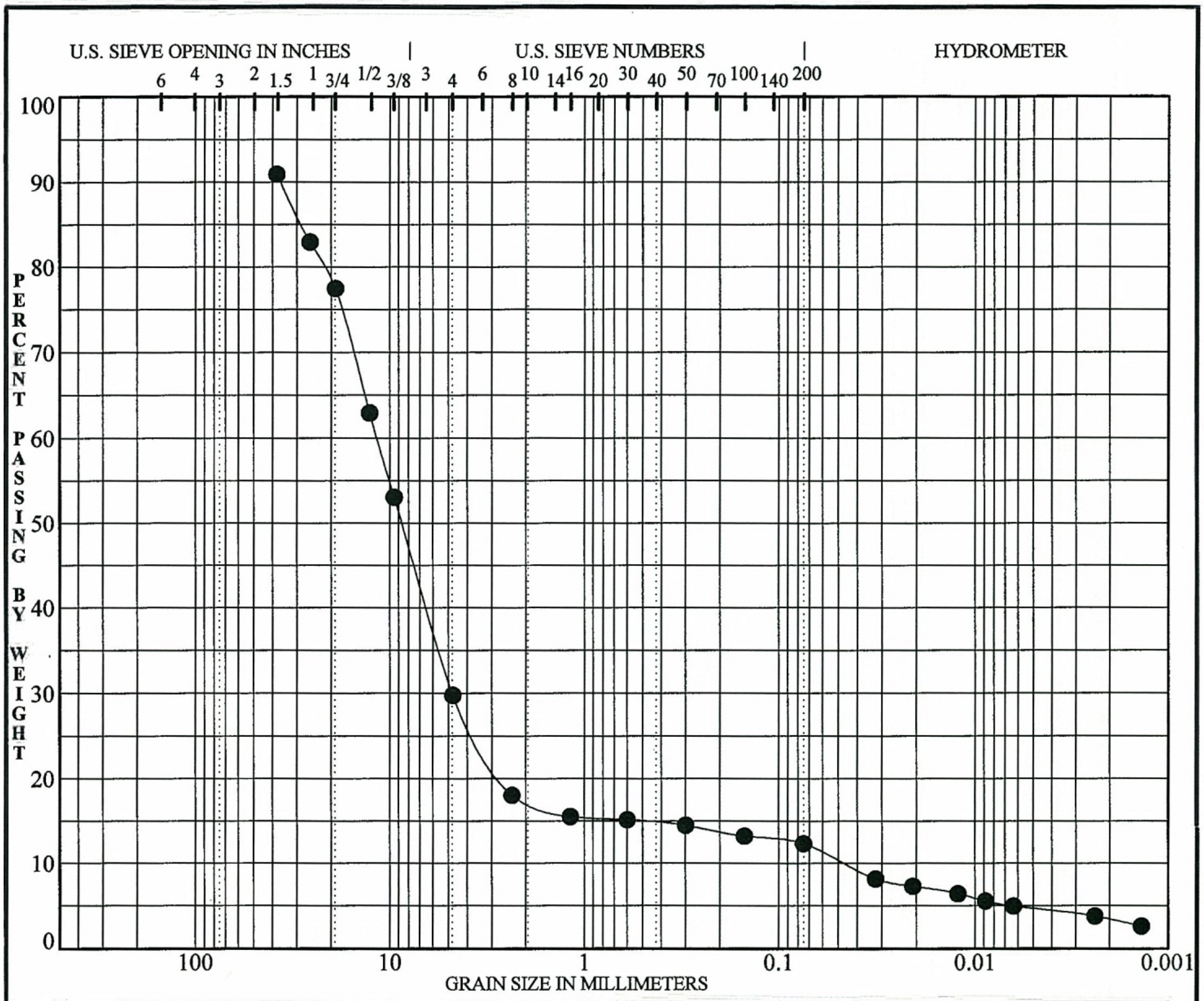
Llagas Creek, California

FIGURE

8

PROJECT No.

590-16



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	09	0.5	12	30	Gray well-graded GRAVEL with silt and sand	GW-GM

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LLAGAS CREEK

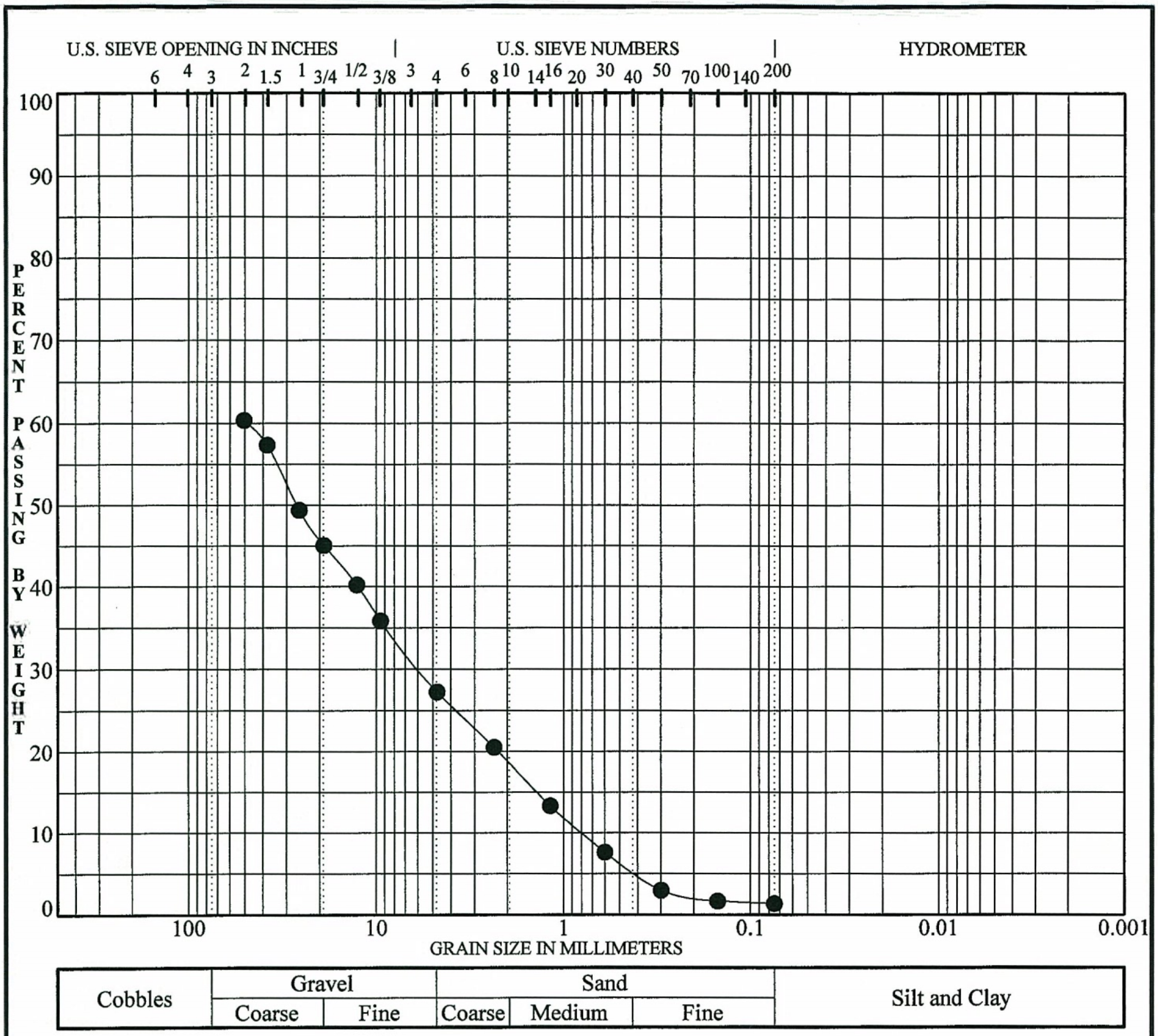
Llagas Creek, California

FIGURE

9

PROJECT No.

590-16



Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	10	0.5	1	27	Gray poorly-graded GRAVEL with sand	GP

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LLAGAS CREEK

Llagas Creek, California

FIGURE

10

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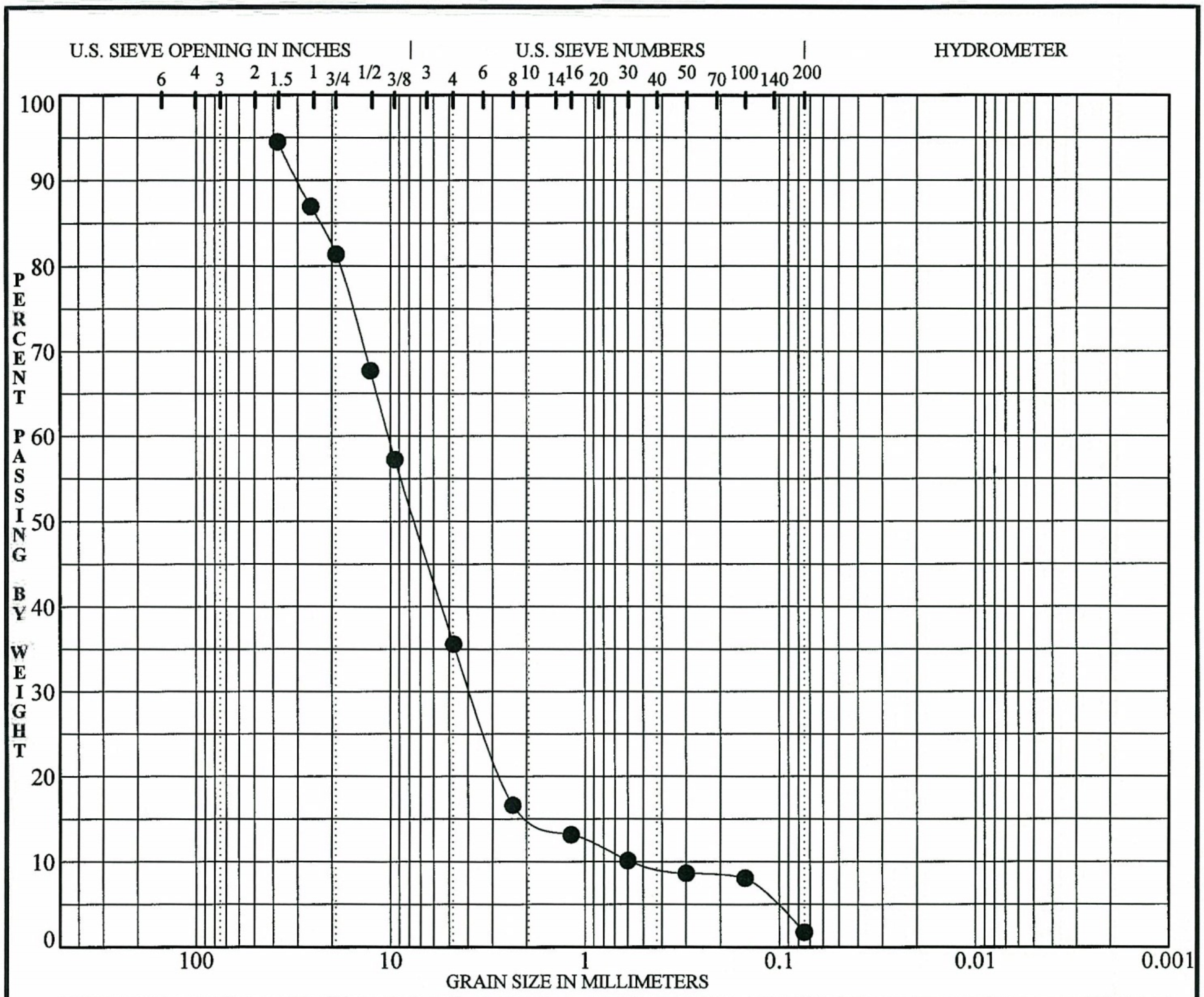

Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	11	0.5	2	36	Gray well-graded GRAVEL with sand	GW

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LLAGAS CREEK
Llagas Creek, California

FIGURE **11**

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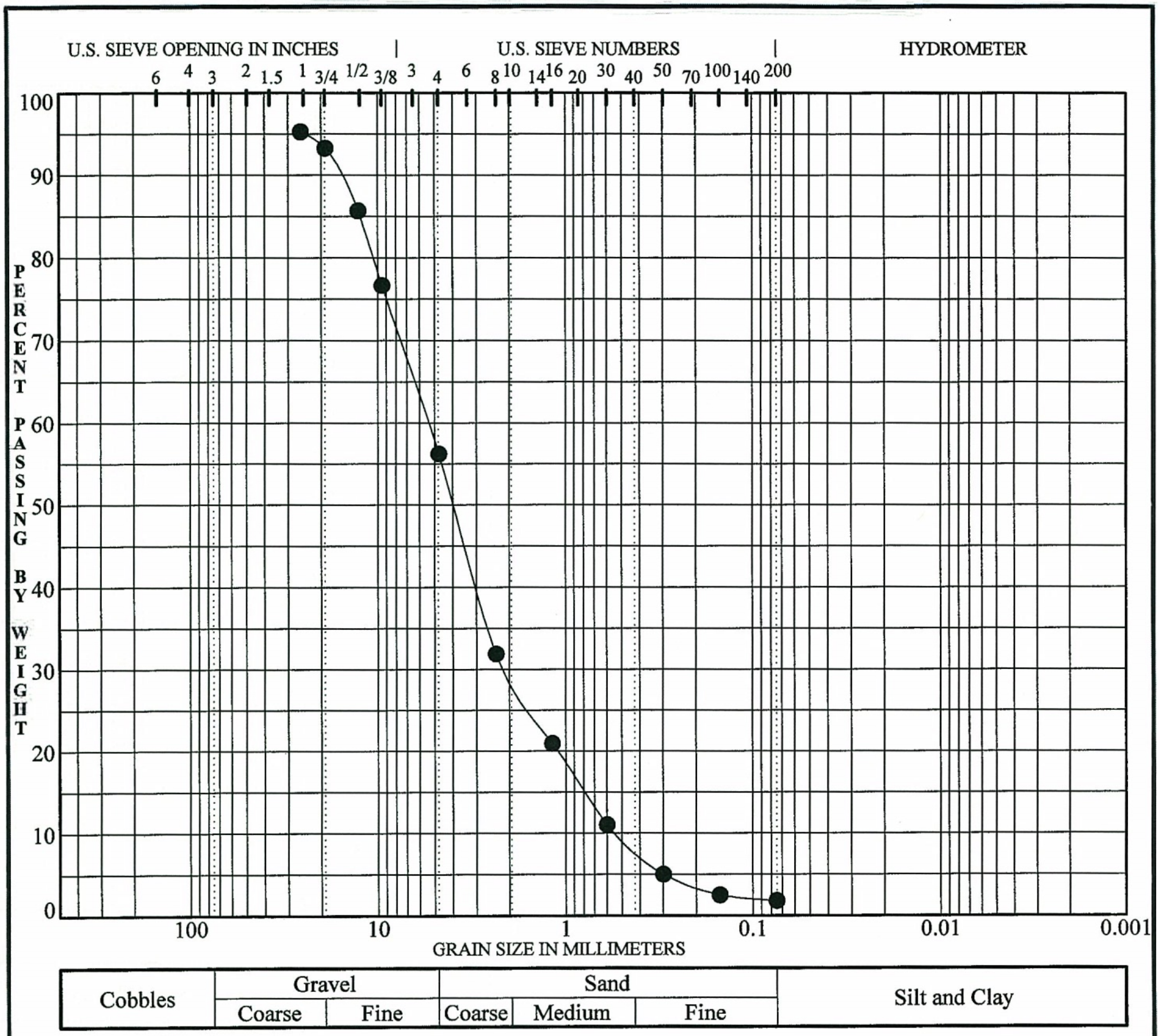

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	12	0.5	2	56	Gray brown well-graded SAND with gravel	SW

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LLAGAS CREEK

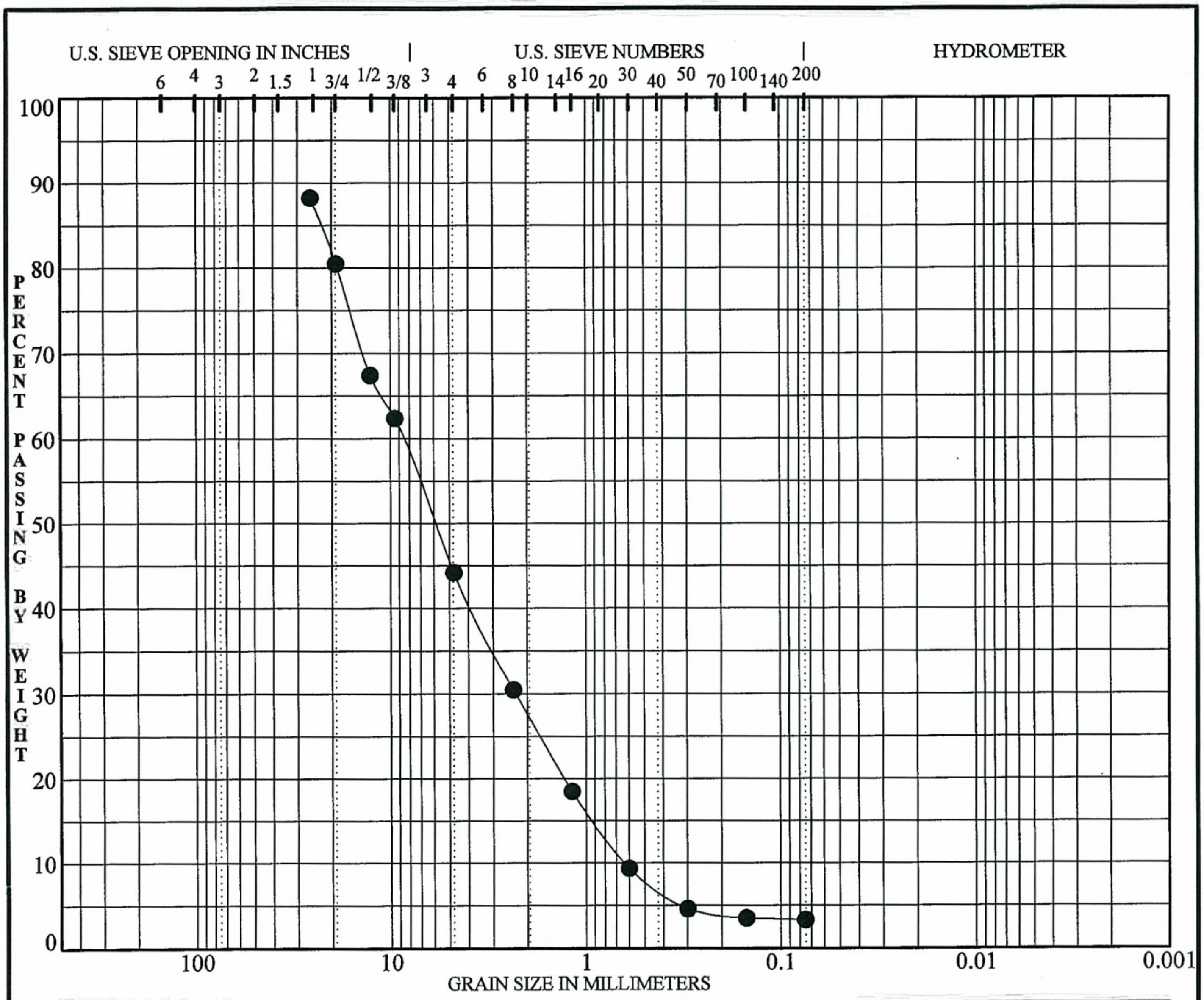
Llagas Creek, California

FIGURE

12

PROJECT No.

590-16



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	13	0.5	3	44	Gray well-graded GRAVEL with sand	GW

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LLAGAS CREEK

Llagas Creek, California

FIGURE

13

PROJECT No.

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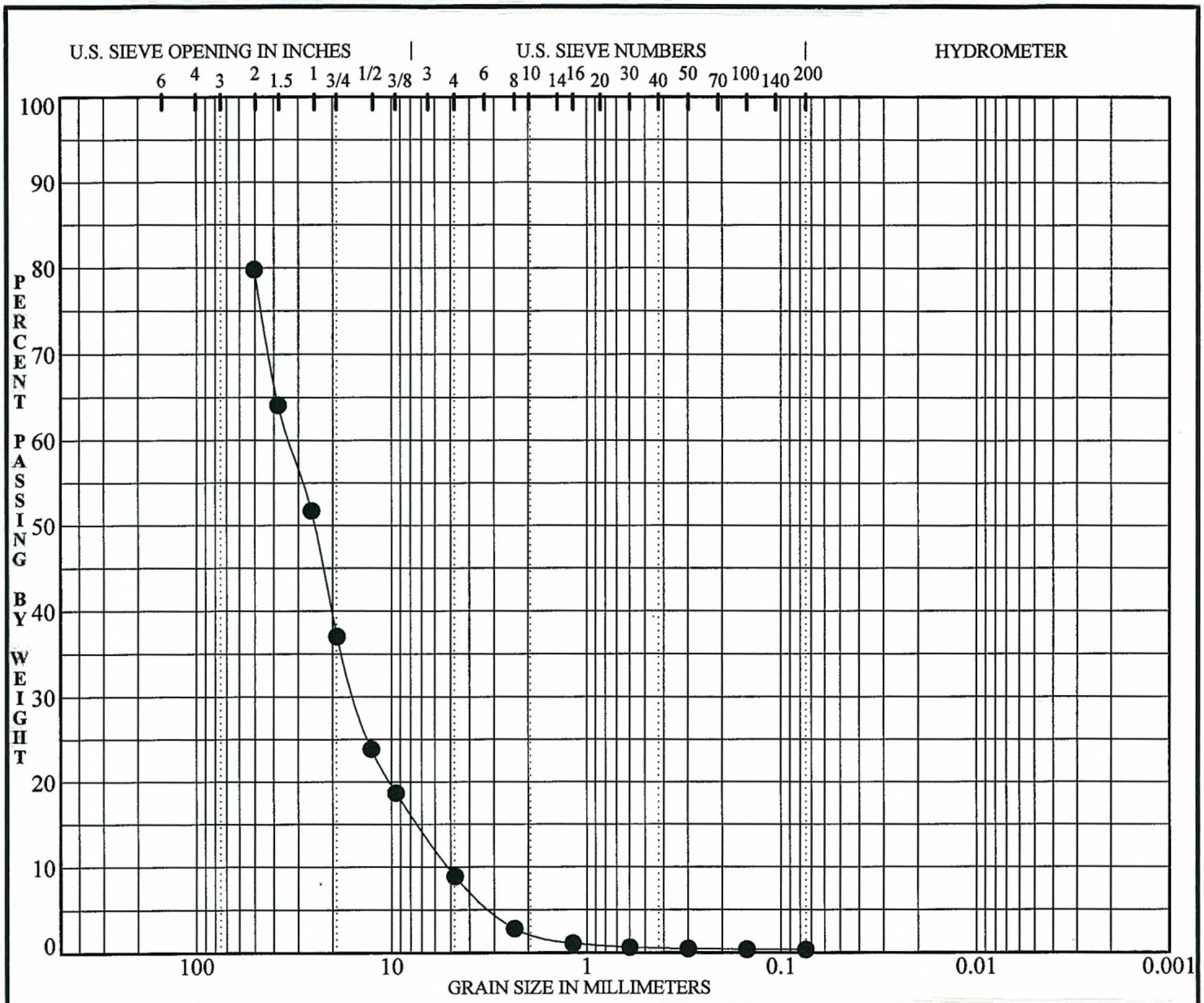

Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	14	0.5	0	9	Gray well-graded GRAVEL	GW

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LLAGAS CREEK

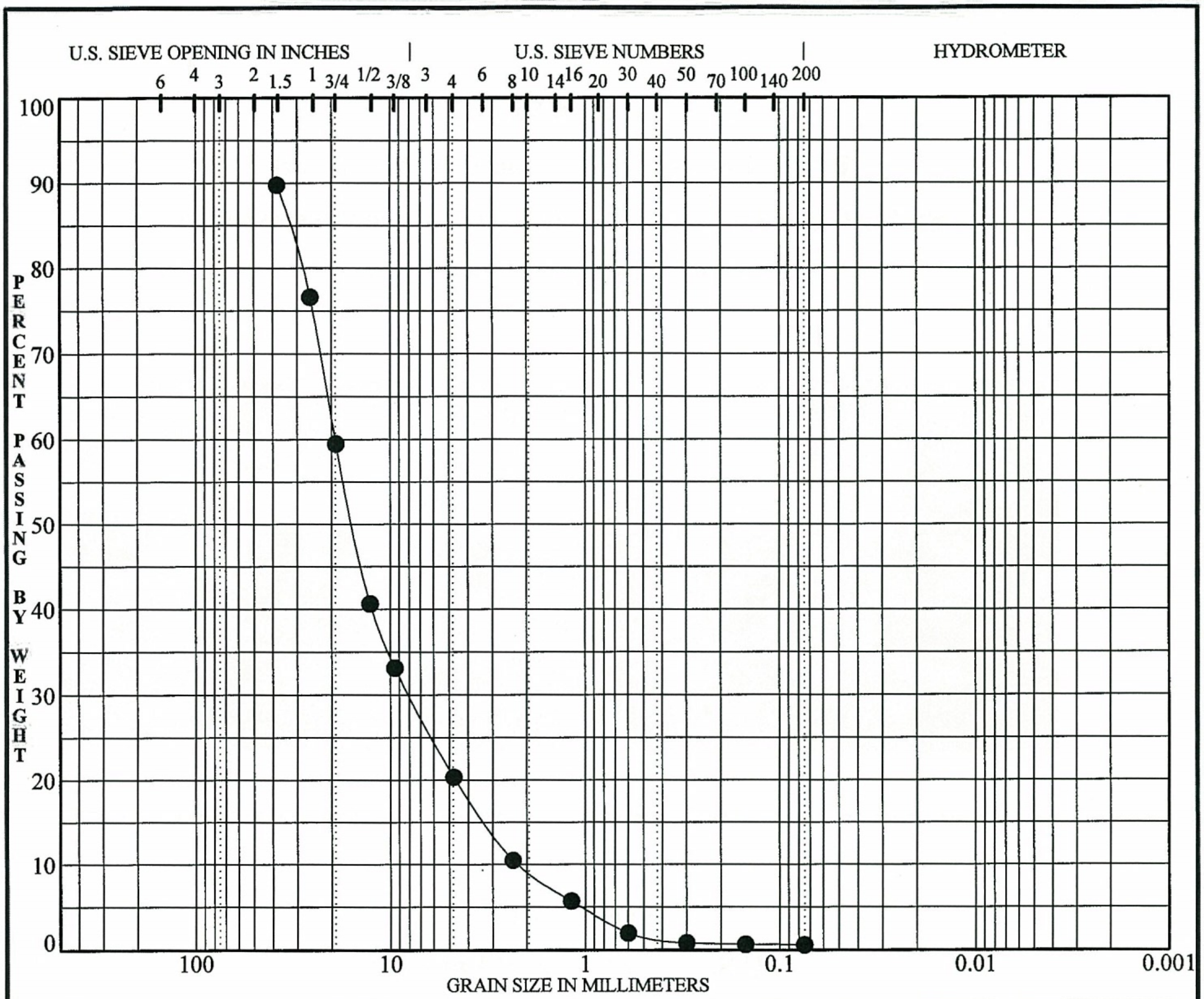
Llagas Creek, California

FIGURE

14

PROJECT No.

590-16



Cobbles	Gravel		Sand			Silt and Clay
	Coarse	Fine	Coarse	Medium	Fine	

Key Symbol	Boring No.	Depth (Feet)	% Passing No. 200 Sieve	% Passing No. 4 Sieve	Sample Descsription	USCS
●	15	0.5	1	20	Gray well-graded GRAVEL with sand	GW

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LLAGAS CREEK

Llagas Creek, California

FIGURE

15

PROJECT No.

590-16

Appendix C

Upper Llagas Creek Flood Control Project

Stable Channel Design and Sediment Impact Assessment

Final Supplementary Report

Prepared For:

U.S. Army Corps of Engineers

San Francisco District

Prepared By:

Noble Consultants, Inc.

Northwest Hydraulic Consultants

January 2008

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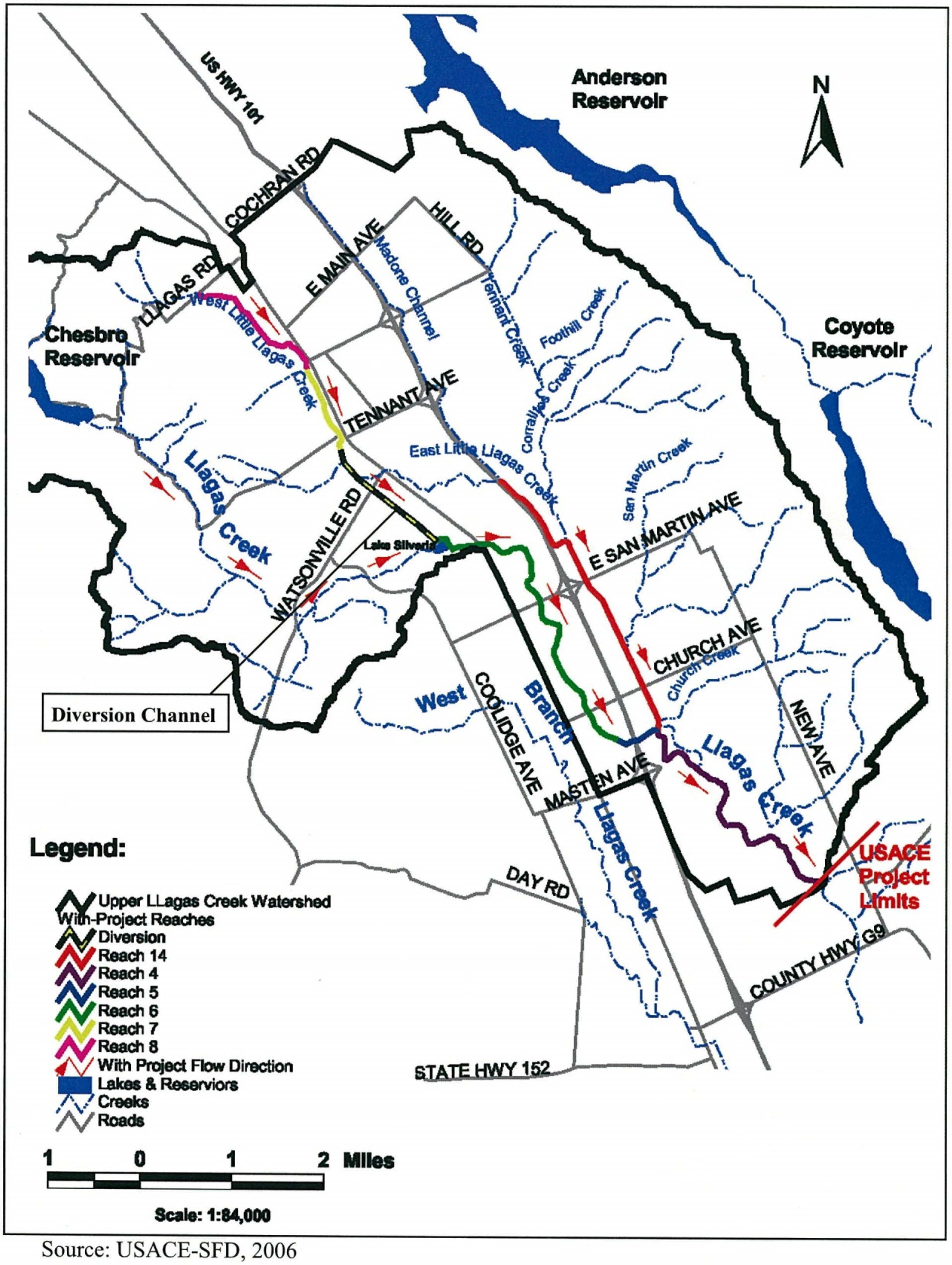
UPPER LLAGAS CREEK FLOOD CONTROL PROJECT
STABLE CHANNEL DESIGN AND SEDIMENT IMPACT ASSESSMENT
SUPPLEMENTARY STUDY

1.0 INTRODUCTION

The Upper Llagas Creek Flood Control Project consists of channel improvements and the construction of an approximately one mile-long diversion channel (USACE-SFD, 2002). The new earthen diversion channel will be located within Reach 7 in the City of Morgan Hill and will connect West Little Llagas Creek (WLLC) to the main branch of Llagas Creek, by bypassing East Little Llagas Creek (ELLC). The proposed diversion channel will connect to Llagas Creek near Silveria Lake located approximately 2,550 feet upstream of the Monterey Highway crossing. **Figure 1-1** illustrates the reach definition in the project area as well as the diversion channel as originally proposed.

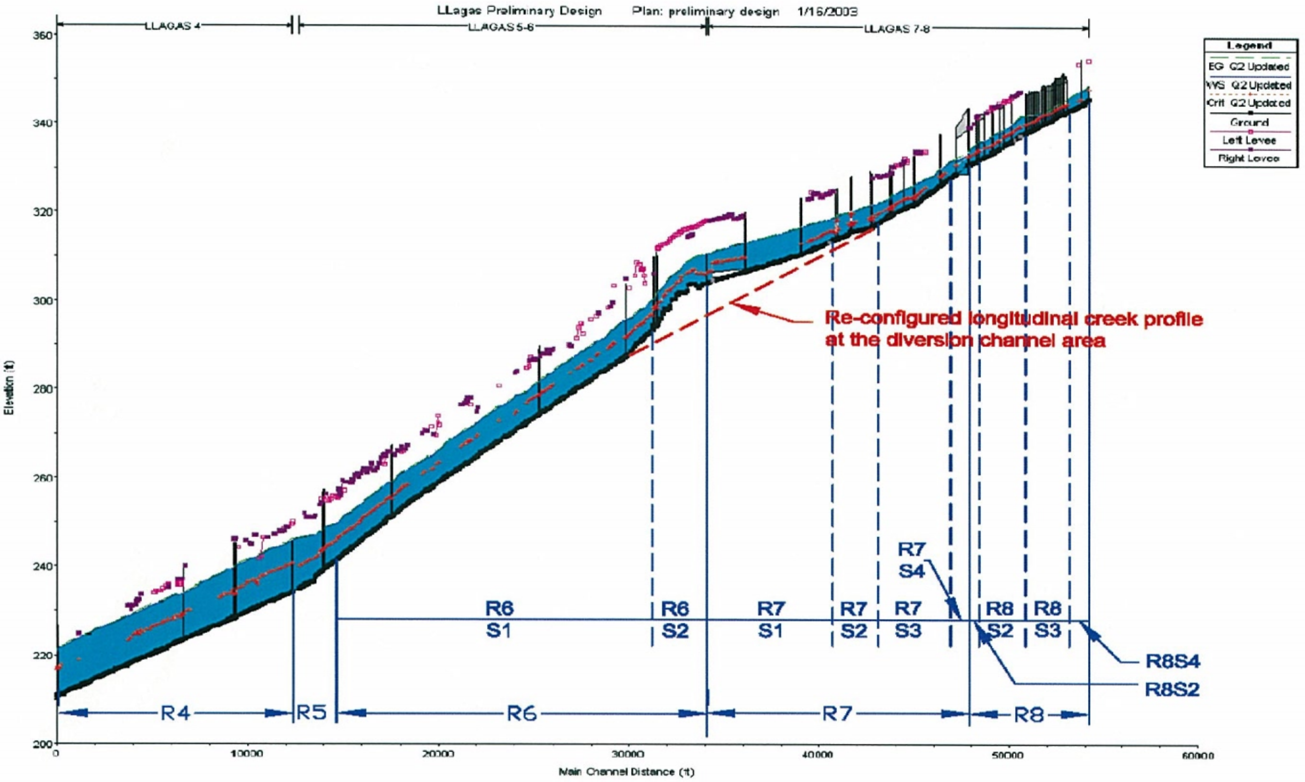
Following the preliminary engineering design (USACE-SFD, 2002), a stable channel design and sediment budget analysis were performed (NCI & **nhc**, 2007). It was determined in the 2007 study that the proposed diversion channel would be fairly unstable owing to its relatively milder longitudinal channel gradient, as shown in **Figure 1-2**. A significant amount of maintenance work will therefore be required to remove the deposited sediment from the diversion channel, as the sediment transport capability of the gentle-gradient diversion channel will be much less than the upstream sediment supply. Two proposed mitigation options were also discussed in the NCI & **nhc** (2007) study to increase the sediment transport capability of the diversion channel and thus alleviate the sedimentation problem. The two options included (1) re-grading the diversion channel to increase the channel gradient, and (2) re-aligning the diversion channel to increase the channel gradient.

The purpose of this supplementary analysis is to formulate a revised diversion channel design that will move sediment through the diversion channel system in an acceptable manner. In addition to the two options initially proposed in the NCI & **nhc** (2007) study, four additional options were identified in this analysis. After the initial screening which eliminated three options based on engineering, environmental and logistic considerations, a sediment budget analysis was performed on three of the newly proposed options as well as the original channel design (NCI & **nhc**, 2007). A preferred design option was then determined based on the results of the computed sediment budget and associated project impacts. The initial and long-term sediment transport conditions of the diversion channel were subsequently investigated and the floodplain mapping was updated for the revised channel design.



Source: USACE-SFD, 2006

Figure 1-1. Reach Schematic for With-Project Conditions



Source: NCI & **nhc**, 2007

Figure 1-2. As-Design and Re-configured Longitudinal Creek Profile

2.0 OPTIONS INVESTIGATION

In addition to the original design and the two mitigation options presented in the previous study (NCI & **nhc**, 2007), four additional options were evaluated. The initial screening process was conducted for all six preliminary design options in order to identify three options for the in-depth sediment budget analysis. The initial screening process was performed jointly with the U.S. Army Corps of Engineers, San Francisco District (USACE-SFD) and the Santa Clara Valley Water District (SCVWD). The evaluation factors used in the screening include the stability of the diversion channel and adjacent reaches in Llagas Creek, the potential environmental and flood impacts, the logistical feasibility, and the construction cost.

The proposed alignments of the diversion channel are shown in **Figure 2-1** for the original design (Option 1) and the six new alternatives (Options 2 to 7). A description of each option is briefly summarized as follows:

*(1) Option 1: The option that was originally formulated in the NCI & **nhc** (2007) study.*

This option will follow the same design of the alignment and longitudinal channel gradient (0.14 percent) that was proposed in the USACE-SFD (2002) study.

(2) Option 2: Sedimentation basin option.

This option will not change the alignment or the longitudinal gradient of the diversion channel that was originally proposed in the USACE-SFD (2002) study. Rather, a sedimentation basin is proposed to trap excessive sediment that would otherwise be deposited in the diversion channel. The sedimentation basin will be built at a location upstream of the diversion channel. It is expected to trap most of the sediment that would enter the diversion channel, and therefore alleviate the sedimentation problem in the diversion channel.

(3) *Option 3: The mitigation Option 1 as presented in the NCI & **nhc** (2007) report.*

Under this option, the diversion channel will be re-graded to a steeper longitudinal gradient of approximately 0.23 percent. The re-grading will start at the beginning section of R7S2, continue through the diversion channel and R6S2, and end at the Llagas Avenue crossing. Table 2-1 presents the definition of the sub-reaches, and the creek profile is shown in Figure 1-2. The steeper bottom slope under this option will result in a greater sediment transport capability and less sediment deposition in the diversion channel than Option 1.

(4) Option 4: Partial re-alignment of the diversion channel.

This option will involve a partial re-alignment of the diversion channel starting from Station 21+00 in R7S1. The diversion channel will connect to the main branch of Llagas Creek at a cross-section (Station 693+00) upstream of the Monterey Road crossing. The upstream segment R7S2, the diversion channel

(R7S1), and the segment of Llagas Creek between the confluence and the downstream Station 667+00 will be re-graded to a steeper gradient of 0.23 percent. Compared to the original design (Option 1), this option will not only increase the sediment transport capacity and alleviate the sedimentation in the diversion channel, but will also avoid the environmentally sensitive segment, R6S2.

Table 2-1. Channel Locations in Reaches 6 and 7

Reach	Sub-reach	Location	River Station
7	R7S4	d/s Ciolino Ave	153+00
	R7S3	d/s Dewitt Creek	137+00
	R7S2	u/s Edmundson Creek	98+00
	R7S1*	u/s Llagas Creek	76+00
6	R6S2	d/s WLLC diversion channel	718+00
	R6S1	u/s East Little Llagas Creek	589+84.25

*: R7S1 is the diversion channel

(5) *Option 5: The mitigation Option 2 as presented in the NCI &* ***nhc*** *(2007) report.*

This option is similar to Option 4 except the confluence between the re-aligned diversion channel and Llagas Creek will be moved to a location downstream of the Union Pacific Railroad crossing. The resulting channel gradient of the diversion channel will also be 0.23 percent.

(6) *Option 6: Partial re-alignment of the diversion channel.*

This option will involve a partial re-alignment of the diversion channel starting from Station 21+00, similar to Options 4 and 5. The re-aligned diversion channel will converge into Llagas Creek further downstream at Station 667+00, which is located approximately 900 feet downstream of the Llagas Avenue crossing. There will be no channel re-grading for Llagas Creek under this option. The resulting longitudinal bottom slope of the diversion channel will be approximately 0.24 percent.

(7) *Option 7: Complete re-alignment of the diversion channel.*

This option will entail the complete re-alignment of the diversion channel to the east side of Monterey Road. The diversion channel will converge into Llagas Creek at the same confluence as in Option 6. There will be no channel re-grading. The resulting longitudinal bottom slope of the diversion channel will be approximately 0.24 percent.

After the initial screening, three mitigation options (Options 2, 3 and 4) were selected from Options 2 through 7 for further in-depth sediment budget analysis.

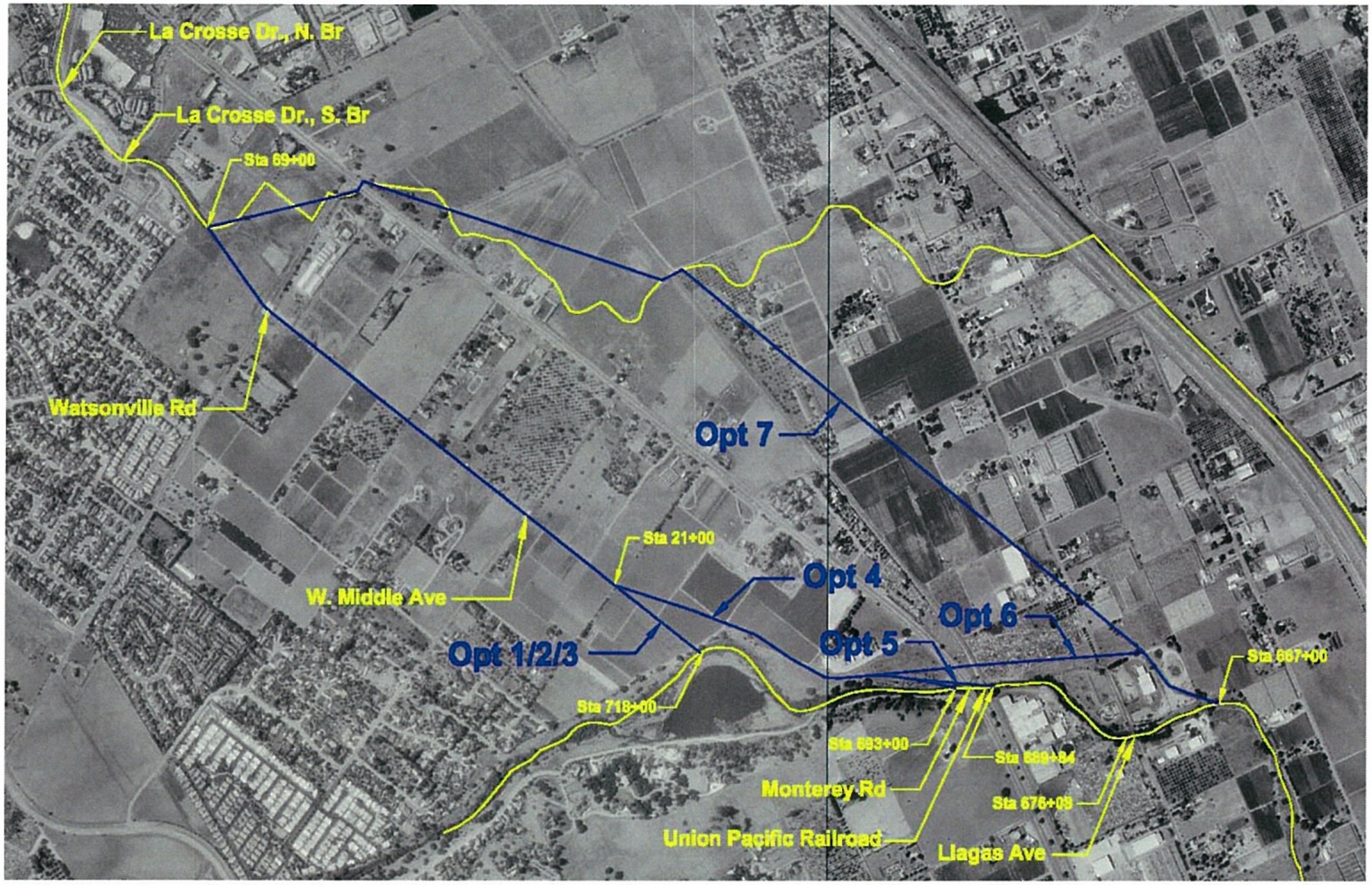


Figure 2-1. Diversion Channel Alignments for Various Options

3.0 SEDIMENT BUDGET ANALYSIS

The sediment budget analysis was conducted for the three selected options (Options 2, 3, and 4) as well as the original channel design (Option 1) as presented in the previous report (NCI & **nhc**, 2007). The purpose of this analysis is to assess the sediment supply/yield conditions within the diversion channel and adjacent reaches for the above-selected options. The four options are summarized in **Table 3-1**, and the channel alignments are shown in **Figure 3-1**. Various trap coefficients of the sediment basin associated with Option 2 were evaluated. It was determined that a trap coefficient of 74 percent would result in the best overall sediment balance in the diversion channel for different flood scenarios. As a result, the entrapping efficiency of the proposed sediment basin in Option 2 was assumed to be 74 percent in this sediment budget analysis.

Table 3-1. Analyzed Options

Option	Description
1	NCI & **nhc** (2007) design
2	Sedimentation basin with a trap coefficient of 74 percent
3	Mitigation option presented in the NCI & **nhc** (2007) study
4	Partial re-aligning the diversion channel

3.1 Sediment Grain Size Distribution

The soil data presented in the geotechnical investigation report prepared by URS (2006) for the Santa Clara Valley Water District was used in the sediment budget analysis. The derived sediment grain size distributions are presented in Appendices A and D of the URS (2006) report. Appendix A lists the results derived by Fugro in the previous investigation, and Appendix D presents the results provided by URS. In total, there are 25 sediment grain size distribution curves presented in these two appendices for Reach 7A, 28 curves for Reach 7B, and 44 curves for Reach 8. Reach 7A begins at La Crosse Street and ends at the confluence with the main branch of Llagas Creek and the diversion channel. Reach 7B is the upstream segment of Reach 7 between West Dunne Avenue and La Crosse Street. The sediment size distribution curves were derived for a range of depths at different boring locations.

The sediment grain size distributions for the 44 samples within Reach 8 are shown as thin grey curves in **Figure 3-2**. The mean distribution averaged over the 44 samples of Reach 8 is shown in this figure as the solid-square line. The sediment grain size distribution curves for the individual soil samples and the derived mean distribution are shown in **Figure 3-3** for Reach 7B and in **Figure 3-4** for Reach 7A.

The sediment grain size distributions show a large scatter range for each of these three reaches. The bed material gradation varies significantly with each boring location and various sampling depths. No discernable trend could be identified. As a result, the mean sediment grain size distribution was used as the representative bed gradation for the sediment budget analysis. This was calculated by taking the average of all the 97 samples within these three reaches. The representative bed material gradation, the mean sediment grain size distributions for the individual sampling reaches (URS, 2006) and the distributions derived in the previous study (NCI & **nhc**, 2007) are shown in **Figure 3-5** for comparison.

It is noted that the average sediment grain size distribution derived from the URS and Fugro boring samples (URS, 2006) is very close to that derived from the previous NCI & **nhc** (2007) bed surface material samples in Reach 8. However, the mean bed material gradation averaged over the URS (2006) boring samples in Reaches 7 and 8 is finer than the mean derived by averaging the samples collected within the entire Llagas Creek system during the previous study (NCI & **nhc,** 2007). A coarser mean bed material gradation was calculated in the previous study (NCI & **nhc,** 2007) because it includes the samples from the downstream reaches (such as Reaches 4, 5 and 6) where a coarser bed gradation generally exists.

3.2 Updated Stable Channel Design for Reach 7

As a result of the channel re-grading and/or re-alignment associated with Options 3 and 4, the longitudinal gradient of the diversion channel (R7S1) and the upstream segment (R7S2) will be increased to approximately 0.23 percent for these two options. This bottom slope is steeper than the original design bottom slope of 0.14 percent to 0.15 percent as presented in the previous NCI & **nhc** (2007) study. Therefore, the channel cross-sections of R7S1 and R7S2 were re-designed in this analysis for Options 3 and 4. In addition, the cross-section of R7S3 was also re-designed for the new sediment grain size distribution.

The method used for the updated stable channel design is the same as that presented in the previous NCI & **nhc** (2007) report. The SAM hydraulic design package for channels (Thomas, W.A., et. al, 2002) was used to calculate a family of solutions of channel slope and depth in terms of width for a given effective discharge. The final depth and width of the stable channel were then determined based on the longitudinal gradient of the creek. The effective discharges that were presented in the previous NCI & **nhc** (2007) study for the three segments (R7S3, R7S2 and R7S1) were also applied to this updated design.

The computed solutions of the stable channel dimensions are shown in **Figure 3-6, Figure 3-7** and **Figure 3-8** for R7S3, R7S2 and R7S1, respectively. The longitudinal channel gradient is approximately 0.24 percent for R7S3, as presented in the NCI & **nhc** (2007) report, and is 0.23 percent for the re-designed diversion channel (R7S1) and R7S2 under Options 3 and 4. The dimensions of the stable low-flow channel were determined for each sub-reach based on these gradients. The results are summarized in **Table 3-2**.

Table 3-2. Updated Dimension of the Low-Flow Channel

Sub-reach	Channel gradient	Low-flow channel			Bench width (ft)
		Width (ft)	Height (ft)	Side slope	
R7S3	0.24%	24	1.6	3:1	Variable
R7S2	0.23%	26	1.8	3:1	Variable
R7S1	0.23%	36	1.7	2.5:1	Variable

The original design depth of the low-flow channel, as presented in the previous NCI & **nhc** (2007) report, is 1.7 feet for the three sub-reaches. The base width varies between 25 feet in R7S3 and 30 feet in R7S2 and R7S1. The re-designed low-flow channels will have similar depths as in the previous design (NCI & **nhc,** 2007) but with different base widths. **Figure 3-9, Figure 3-10** and **Figure 3-11** show the representative low-flow channel geometries in the individual sub-reaches, compared with the preliminary design (USACE-SFD, 2002).

3.3 Sediment Yield

The sediment yield was calculated for the four selected options (Options 1, 2, 3, and 4), based on the updated bed material gradation and the modified channel dimension. The sediment yield was computed for (1) the average annual conditions, (2) effective discharge, (3) ten-percent, and (4) one-percent chance exceedance flood events using the flow duration curves and the synthetic hydrographs as determined in the previous study (NCI & **nhc,** 2007). It is noted that the average annual conditions represent the cumulative flow discharge over an entire year, while the discharge conditions 2, 3 and 4 are the individual flood events. The sediment transport module of the SAM package was used to calculate the sediment discharge rating curves, and the sediment yield module was used to determine the sediment yield for various flood events.

The sediment budget analysis was only conducted for the four sub-reaches within Reach 7 and the two sub-reaches within Reach 6 as the re-design of the diversion channel will mainly impact the diversion channel and the reaches immediately adjacent to it. The computed sediment yield for various flood events is summarized in **Table 3-3** for Options 1 and 2, in **Table 3-4** for Option 3, and in **Table 3-5** for Option 4.

The sediment yield is identical for Options 1 and 2 as they share the same channel geometry and longitudinal gradient. The sediment yield of the diversion channel (R7S1) and sub-reach R7S2 for Options 3 and 4 will be significantly greater than the previous NCI & **nhc** (2007) design (Option 1) as a result of the increased longitudinal gradient. The opposite will occur in Sub-reach R6S2 for Options 3 and 4. Option 3 will result in a milder longitudinal gradient for R6S2, compared to Option 1, as a result of channel re-grading in this sub-reach. Option 4 will result in less water yield for R6S2 compared to Option 1 because of the extension of the diversion channel which bypasses the major portion of R6S2. Therefore, Options 3 and 4 will result in less sediment yield in Sub-reach R6S2.

Table 3-3. Computed Sediment Yield for Options 1 and 2

Flow Event	Sub-reach	Water Yield (ac-ft)	Sediment Yield					
			Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)	Mean (cu yd)
A	R7S4	2,201	1,465	505	3,415	1,711	1,442	1,148
	R7S3	2,906	2,205	857	5,741	2,577	2,299	1,831
	R7S2	3,479	661	129	1,363	1,043	590	470
	R7S1	4,273	934	218	2,150	1,463	895	713
	R6S2	6,705	5,912	424	33,199	9,550	5,310	4,229
	R6S1	6,705	6,053	641	31,031	8,941	5,728	4,562
B	R7S4	109	151	58	366	167	152	121
	R7S3	140	223	100	627	250	243	194
	R7S2	172	68	12	146	99	59	47
	R7S1	211	95	20	226	137	88	70
	R6S2	346	552	48	3,428	893	534	425
	R6S1	346	530	72	3,239	799	561	447
C	R7S4	659	1,610	1,380	11,524	2,294	2,768	2,205
	R7S3	868	2,097	1,964	16,527	2,990	3,777	3,008
	R7S2	966	667	475	3,599	1,116	1,062	846
	R7S1	1,248	958	790	5,972	1,655	1,654	1,317
	R6S2	3,013	7,203	1,620	118,924	16,106	12,227	9,739
	R6S1	3,013	6,187	2,358	101,962	12,072	11,576	9,220
D	R7S4	1,446	4,504	4,577	44,089	7,044	8,945	7,125
	R7S3	1,899	5,566	6,161	61,207	8,753	11,642	9,273
	R7S2	2,022	1,768	1,650	12,442	3,141	3,268	2,603
	R7S1	2,681	2,613	2,791	21,554	4,830	5,249	4,181
	R6S2	7,085	24,980	9,561	707,129	45,694	52,706	41,980
	R6S1	7,085	15,998	7,700	375,737	35,208	35,729	28,458

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

Table 3-4. Computed Sediment Yield for Option 3

Flow Event	Sub-reach	Water Yield (ac-ft)	Sediment Yield					
			Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)	Mean (cu yd)
A	R7S4	2,201	1,465	505	3,415	1,711	1,442	1,148
	R7S3	2,906	2,235	873	5,873	2,619	2,341	1,864
	R7S2	3,479	1,634	411	4,178	2,255	1,586	1,263
	R7S1	4,273	2,070	595	5,496	2,819	2,090	1,665
	R6S2	6,705	2,356	81	10,268	4,174	1,691	1,347
	R6S1	6,705	6,053	641	31,031	8,941	5,728	4,562
B	R7S4	109	151	58	366	167	152	121
	R7S3	140	225	103	649	254	249	198
	R7S2	172	150	37	448	211	151	121
	R7S1	211	194	50	556	262	194	154
	R6S2	346	230	9	1,082	392	172	137
	R6S1	346	530	72	3,239	799	561	447
C	R7S4	659	1,610	1,380	11,524	2,294	2,768	2,205
	R7S3	868	2,056	1,932	16,325	2,938	3,715	2,959
	R7S2	966	1,428	1,069	9,940	2,321	2,436	1,940
	R7S1	1,248	2,111	1,834	16,597	3,441	3,856	3,071
	R6S2	3,013	3,070	486	35,088	6,677	4,324	3,444
	R6S1	3,013	6,187	2,358	101,962	12,072	11,576	9,220
D	R7S4	1,446	4,504	4,577	44,089	7,044	8,945	7,125
	R7S3	1,899	5,425	6,023	60,179	8,558	11,390	9,072
	R7S2	2,022	3,728	3,432	33,932	6,509	7,291	5,807
	R7S1	2,681	5,761	6,108	61,278	10,259	12,196	9,714
	R6S2	7,085	9,195	2,231	120,039	19,626	14,827	11,810
	R6S1	7,085	15,998	7,700	375,737	35,208	35,729	28,458

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

Table 3-5. Computed Sediment Yield for Option 4

Flow Event	Sub-reach	Water Yield (ac-ft)	Sediment Yield					
			Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)	Mean (cu yd)
A	R7S4	2,201	1,465	505	3,415	1,711	1,442	1,148
	R7S3	2,906	2,235	873	5,873	2,619	2,341	1,864
	R7S2	3,479	1,634	411	4,178	2,255	1,586	1,263
	R7S1	4,273	2,070	595	5,496	2,819	2,090	1,665
	R6S2	2,432	2,144	154	12,042	3,464	1,926	1,534
	R6S1	6,705	6,053	641	31,031	8,941	5,728	4,562
B	R7S4	109	151	58	366	167	152	121
	R7S3	140	225	103	649	254	249	198
	R7S2	172	150	37	448	211	151	121
	R7S1	211	194	50	556	262	194	154
	R6S2	135	215	19	1,338	348	208	166
	R6S1	346	530	72	3,239	799	561	447
C	R7S4	659	1,610	1,380	11,524	2,294	2,768	2,205
	R7S3	868	2,056	1,932	16,325	2,938	3,715	2,959
	R7S2	966	1,428	1,069	9,940	2,321	2,436	1,940
	R7S1	1,248	2,111	1,834	16,597	3,441	3,856	3,071
	R6S2	1,765	4,219	949	69,665	9,435	7,163	5,705
	R6S1	3,013	6,187	2,358	101,962	12,072	11,576	9,220
D	R7S4	1,446	4,504	4,577	44,089	7,044	8,945	7,125
	R7S3	1,899	5,425	6,023	60,179	8,558	11,390	9,072
	R7S2	2,022	3,728	3,432	33,932	6,509	7,291	5,807
	R7S1	2,681	5,761	6,108	61,278	10,259	12,196	9,714
	R6S2	4,404	15,527	5,943	439,548	28,403	32,762	26,095
	R6S1	7,085	15,998	7,700	375,737	35,208	35,729	28,458

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

3.4 Sediment Budget

The sediment supply (S) and the sediment yield (Y) were calculated for the re-grouped four sub-reaches (R7S4/R7S3, R7S2/R7S1, R6S2, and R6S1) for the four analyzed options using the computed sediment yield of each sun-reach. The methodology for calculating the sediment budget is described in Section 5.4.3 of the previous NCI & **nhc** (2007) report. The computed sediment supply, sediment yield, and the resulting sediment balance and creek stability for each sub-reach are summarized in **Table 3-6** through **Table 3-9** for each of the four options. The channel is considered stable if the difference between the sediment supply and the sediment yield is within 15 percent.

Table 3-6. Computed Sediment Budget for Option 1

Flow Event	Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S-Y (tons)	$\frac{S-Y}{S}$	Stability
				Supply (S)	Yield (Y)			
A	R7S4, R7S3	5,040	2,906	2,254	2,299	-45	-2%	Stable
	R7S2, R7S1	8,600	4,273	2,586	895	1,691	65%	Deposition
	R6S2	2,900	6,705	2,820	5,310	-2,489	-88%	Erosion
	R6S1	16,400	6,705	5,310	5,728	-418	-8%	Stable
B	R7S4, R7S3	5,040	140	233	243	-10	-4%	Stable
	R7S2, R7S1	8,600	211	273	88	185	68%	Deposition
	R6S2	2,900	346	296	534	-238	-80%	Erosion
	R6S1	16,400	346	534	561	-27	-5%	Stable
C	R7S4, R7S3	5,040	868	4,232	3,777	455	11%	Stable
	R7S2, R7S1	8,600	1,248	4,281	1,654	2,627	61%	Deposition
	R6S2	2,900	3,013	8,816	12,227	-3,411	-39%	Erosion
	R6S1	16,400	3,013	12,227	11,576	651	5%	Stable
D	R7S4, R7S3	5,040	1,899	13,650	11,642	2,007	15%	Stable
	R7S2, R7S1	8,600	2,681	13,173	5,249	7,924	60%	Deposition
	R6S2	2,900	7,085	38,011	52,706	-14,695	-39%	Erosion
	R6S1	16,400	7,085	52,706	35,729	16,977	32%	Deposition

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

Table 3-7. Computed Sediment Budget for Option 2

Flow Event	Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S-Y (tons)	$\frac{S-Y}{S}$	Stability
				Supply (S)	Yield (Y)			
A	R7S4, R7S3	5,040	2,906	2,254	2,299	-45	-2%	Stable
	R7S2, R7S1	8,600	4,273	884	895	-11	-1%	Stable
	R6S2	2,900	6,705	2,820	5,310	-2,489	-88%	Erosion
	R6S1	16,400	6,705	5,310	5,728	-418	-8%	Stable
B	R7S4, R7S3	5,040	140	233	243	-10	-4%	Stable
	R7S2, R7S1	8,600	211	93	88	5	6%	Stable
	R6S2	2,900	346	296	534	-238	-80%	Erosion
	R6S1	16,400	346	534	561	-27	-5%	Stable
C	R7S4, R7S3	5,040	868	4,232	3,777	455	11%	Stable
	R7S2, R7S1	8,600	1,248	1486	1,654	-168	-11%	Stable
	R6S2	2,900	3,013	8,816	12,227	-3,411	-39%	Erosion
	R6S1	16,400	3,013	12,227	11,576	651	5%	Stable
D	R7S4, R7S3	5,040	1,899	13,650	11,642	2,007	15%	Stable
	R7S2, R7S1	8,600	2,681	4558	5,249	-691	-15%	Stable
	R6S2	2,900	7,085	38,011	52,706	-14,695	-39%	Erosion
	R6S1	16,400	7,085	52,706	35,729	16,977	32%	Deposition

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

Table 3-8. Computed Sediment Budget for Option 3

Flow Event	Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S-Y (tons)	$\frac{S-Y}{S}$	Stability
				Supply (S)	Yield (Y)			
A	R7S4, R7S3	5,040	2,906	2,264	2,341	-77	-3%	Stable
	R7S2, R7S1	8,600	4,273	3,009	2,090	919	31%	Deposition
	R6S2	2,900	6,705	2,703	1,691	1,012	37%	Deposition
	R6S1	16,400	6,705	1,691	5,728	-4,037	-239%	Erosion
B	R7S4, R7S3	5,040	140	235	249	-14	-6%	Stable
	R7S2, R7S1	8,600	211	314	194	120	38%	Deposition
	R6S2	2,900	346	261	172	89	34%	Deposition
	R6S1	16,400	346	172	561	-388	-226%	Erosion
C	R7S4, R7S3	5,040	868	4,217	3,715	502	12%	Stable
	R7S2, R7S1	8,600	1,248	4,889	3,856	1,033	21%	Deposition
	R6S2	2,900	3,013	6,389	4,324	2,065	32%	Deposition
	R6S1	16,400	3,013	4,324	11,576	-7,252	-168%	Erosion
D	R7S4, R7S3	5,040	1,899	13,589	11,390	2,200	16%	Stable
	R7S2, R7S1	8,600	2,681	14,947	12,196	2,751	18%	Deposition
	R6S2	2,900	7,085	21,412	14,827	6,585	31%	Deposition
	R6S1	16,400	7,085	14,827	35,729	-20,902	-141%	Erosion

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

Table 3-9. Computed Sediment Budget for Option 4

Flow Event	Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S-Y (tons)	$\frac{S-Y}{S}$	Stability
				Supply (S)	Yield (Y)			
A	R7S4, R7S3	5,040	2,906	2,264	2,341	-77	-3%	Stable
	R7S2, R7S1	8,600	4,273	3,009	2,090	919	31%	Deposition
	R6S2	2,900	2,432	1,926	1,926	0	0%	Stable
	R6S1	16,400	6,705	4,016	5,728	-1,712	-43%	Erosion
B	R7S4, R7S3	5,040	140	235	249	-14	-6%	Stable
	R7S2, R7S1	8,600	211	314	194	120	38%	Deposition
	R6S2	2,900	135	208	208	0	0%	Stable
	R6S1	16,400	346	402	561	-158	-39%	Erosion
C	R7S4, R7S3	5,040	868	4,217	3,715	502	12%	Stable
	R7S2, R7S1	8,600	1,248	4,889	3,856	1,033	21%	Deposition
	R6S2	2,900	1,765	7,163	7,163	0	0%	Stable
	R6S1	16,400	3,013	11,019	11,576	-557	-5%	Stable
D	R7S4, R7S3	5,040	1,899	13,589	11,390	2,200	16%	Stable
	R7S2, R7S1	8,600	2,681	14,947	12,196	2,751	18%	Deposition
	R6S2	2,900	4,404	32,762	32,762	0	0%	Stable
	R6S1	16,400	7,085	44,958	35,729	9,229	21%	Deposition

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

The computed results indicate that the trap efficiency of the diversion channel (represented by the R7S2/R7S1 segment) varies between different flood events and between the four design options. For the average annual conditions (Flood Event A), the trap efficiency will decrease from 65 percent for Option 1 to 31 percent for Options 3 and 4, and the sediment supply will be slightly less than the sediment yield for Option 2. Under the effective discharge scenario (Flow Event B in the tables), the trap efficiency of the diversion channel will be 68 percent for the original design (Option 1), 6 percent for Option 2, and 38 percent for Options 3 and 4. For the one-percent and ten-percent flood events (Flood Events C and D), while the trap efficiency will be approximately 60 percent for Option 1, it will vary between 18 to 21 percent for Options 3 and 4, and the sediment yield will be 11 to 15 percent greater than the sediment supply (negative trap efficiency) for Option 2. The trap efficiency of the diversion channel for Options 2, 3 and 4 will be significantly smaller than Option 1 for all the flood scenarios. In other words, Options 2, 3 and 4 will all reduce the sediment deposition problem in the diversion channel associated with Option 1.

Each design option will impact the creek stability differently in the areas adjacent to the diversion channel, particularly in reaches downstream of the diversion channel. While the sedimentation basin proposed in Option 2 will reduce the sediment supply to the lower portion of Reach 7 (R7S2 and R7S1), the sediment yield of the diversion channel and thus the sediment supply to Reach 6 will be identical for Option 1 and Option 2. Therefore, the sedimentation pattern of Reach 6 will be the same for these two options. Options 3 and 4 will change the sediment supply and/or the sediment transport potential of R6S2 and R6S1, and will therefore change the sedimentation pattern of these two sub-reaches.

These four options were qualitatively ranked in terms of the stability of the diversion channel (R6S2 and R6S1), based on the results of the sediment budget analysis. The ranking scores are summarized in **Table 3-10**. The highest score (4) suggests the most favorable option. A tie between two options is noted as the average of the two consecutive ranking scores (e.g., (2+3)/2 = 2.5). Also presented in this table are the rankings of the options in terms of their environmental impact, flood impact to upstream reaches, and the logistic effort that would be involved in implementation. It should be noted that the rankings in Table 3-10 only present a qualitative assessment for each option, and that the ranking criterion may not be equally weighted. After reviewing the overall computed sediment balance within the subject sub-reaches, it was determined by the Corps that Option 4 is the preferred option. The initial and long-term channel conditions were evaluated for Option 4 only and are presented in the next section.

Table 3-10. Qualitative Rankings of Options

Options	Option 1	Option 2	Option 3	Option 4
Channel Stability of Diversion Channel	1	4	2.5	2.5
Channel Stability of R6S2	2.5	2.5	3	4
Channel Stability of R6S1	3.5	3.5	1	2
Least Environmental Impact to R6S2	2.5	2.5	3	4
Least Flood Impact to Upstream Reach	2	1	3.5	3.5
Least Logistic Effort	4	3	2	1

Note: Score 4 is the highest (most favorable); Score 1 in the lowest (least favorable)

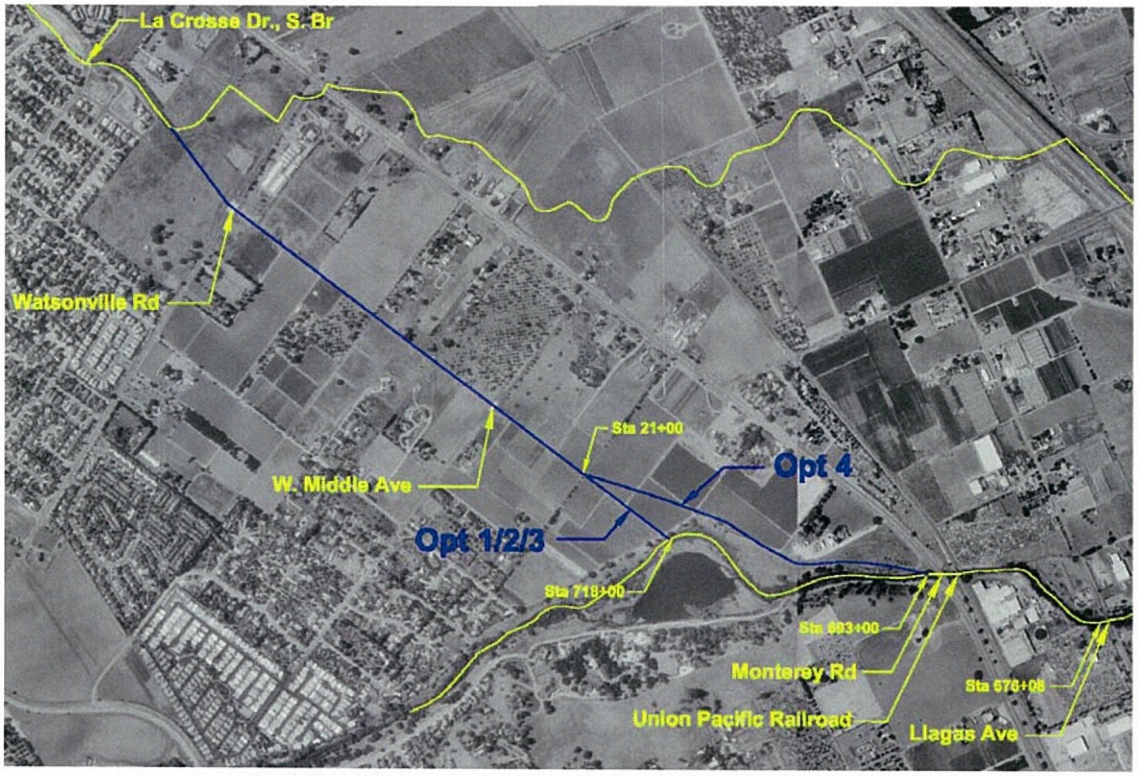


Figure 3-1. Diversion Channel Alignments for the Selected Options

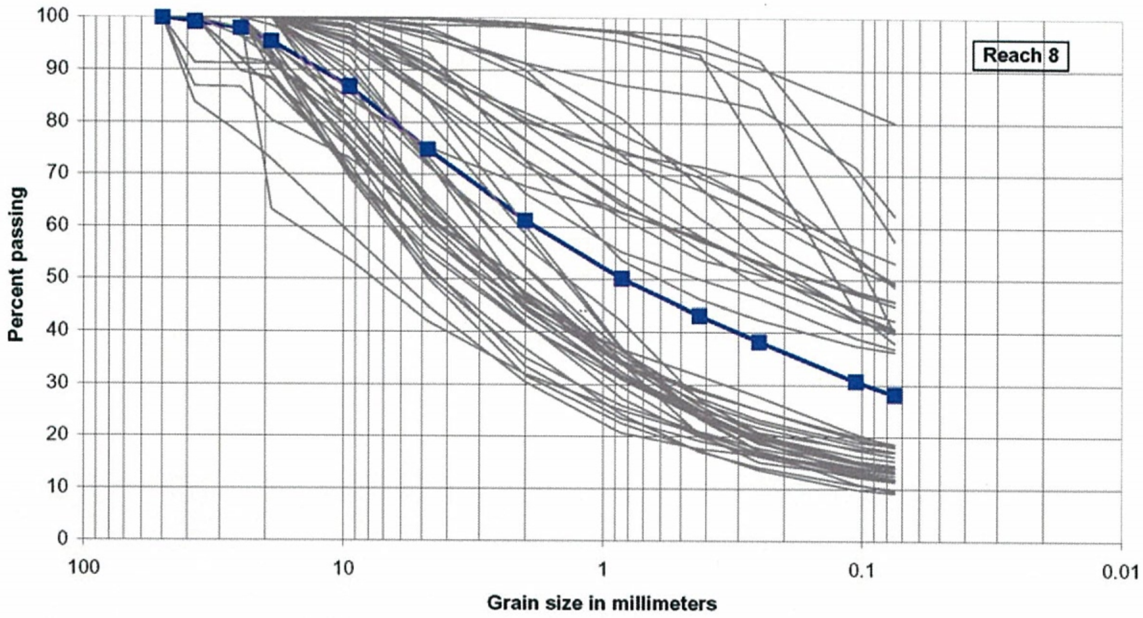


Figure 3-2. Sediment Grain Size Distributions for Reach 8

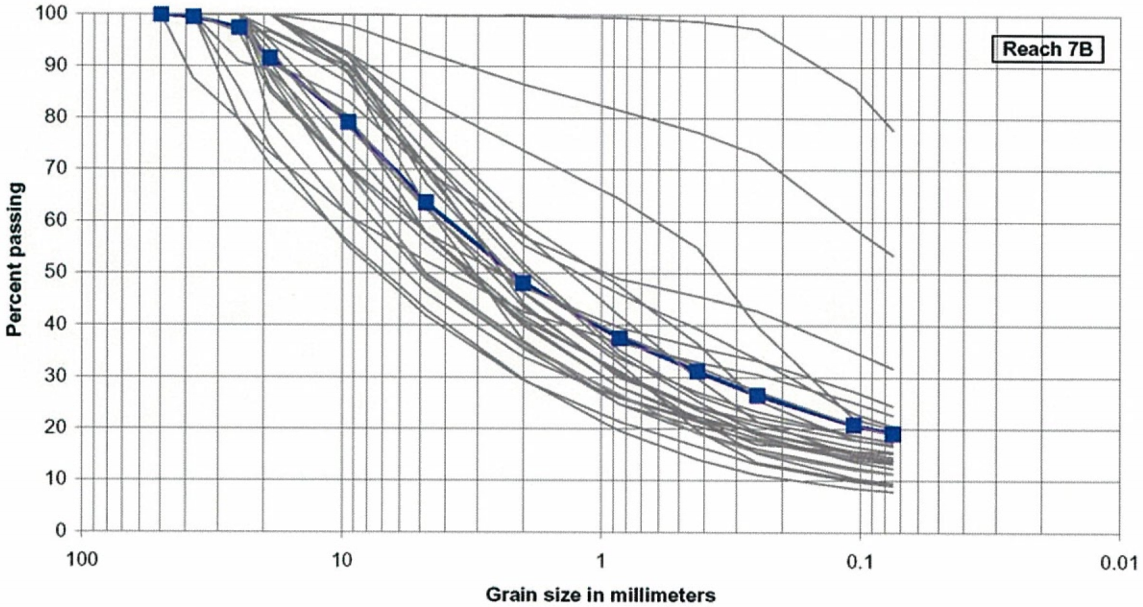


Figure 3-3. Sediment Grain Size Distributions for Reach 7B

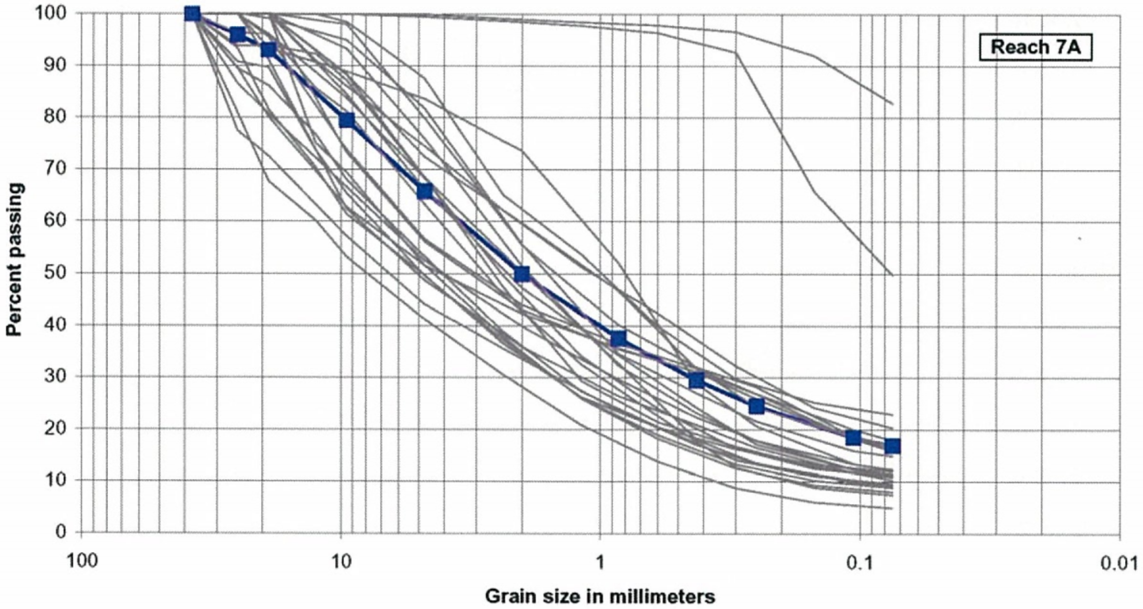
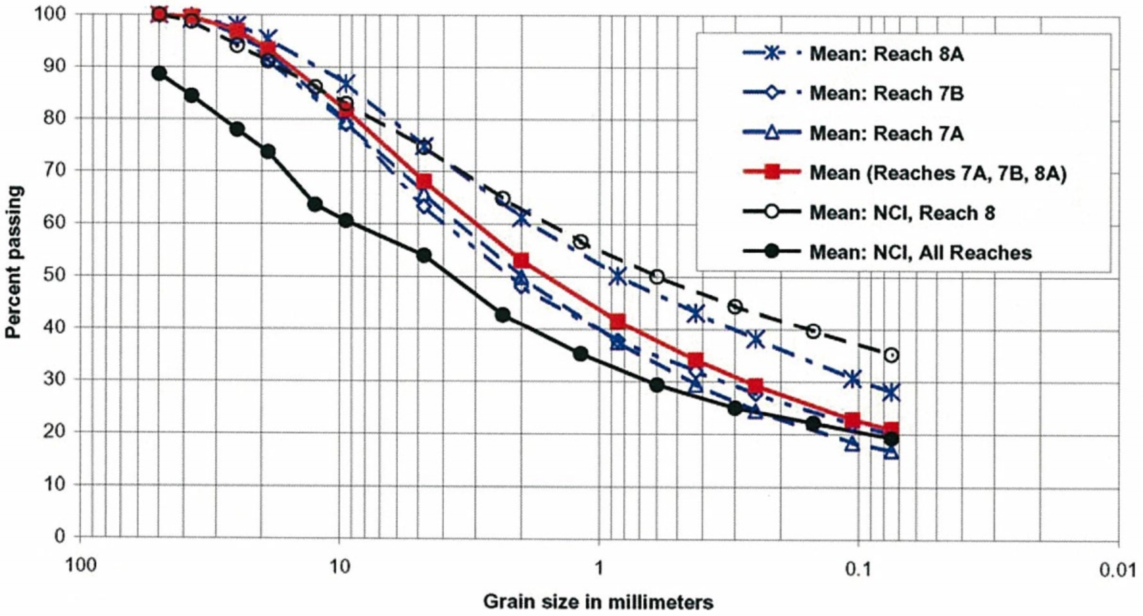


Figure 3-4. Sediment Grain Size Distributions for Reach 7A



Note: Reach 8A defined in the URS report is approximately in the same reach boundary as Reach 8 defined in the USACE-SFD 2002 report.

Figure 3-5. Averaged Sediment Grain Size Distributions

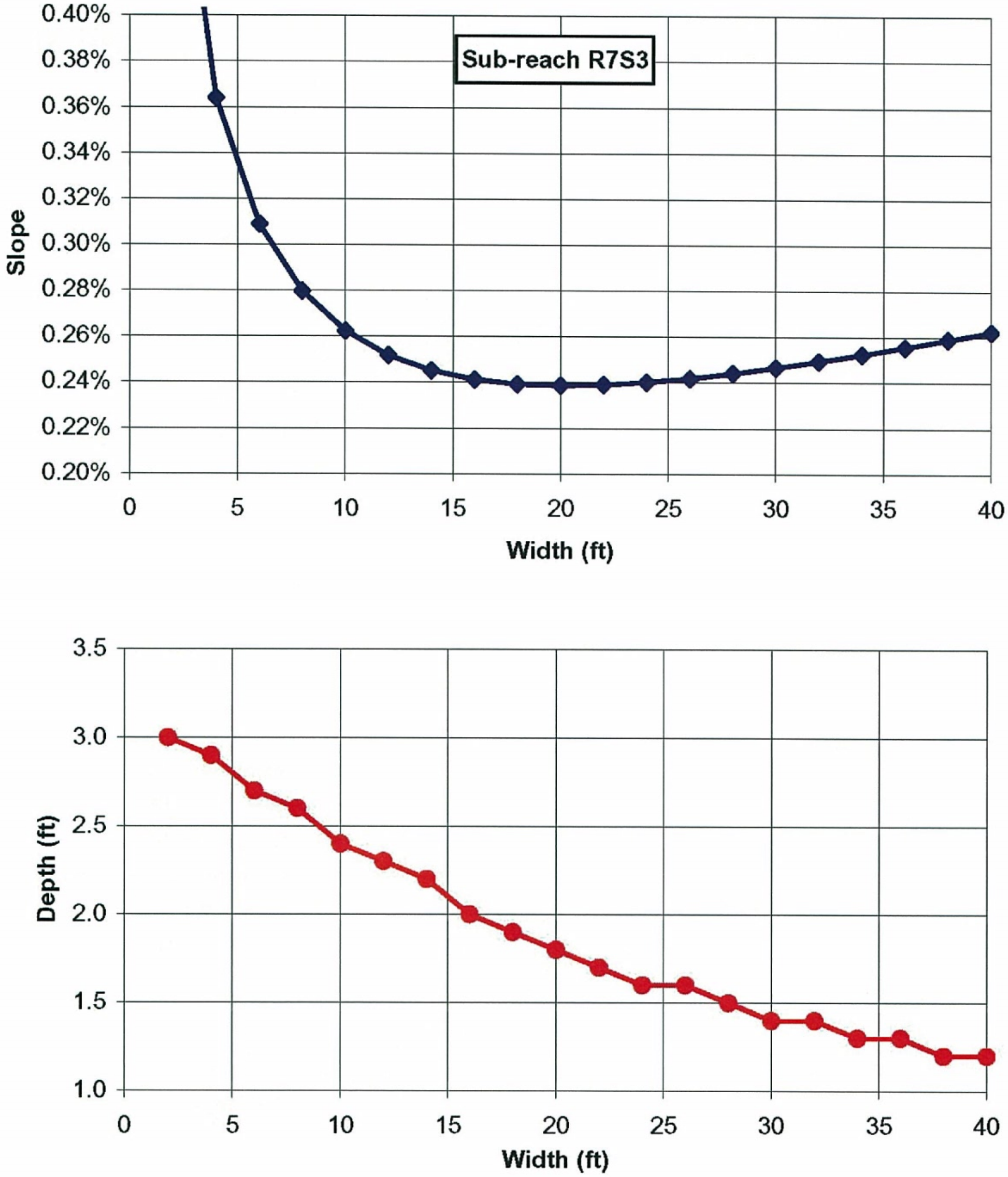


Figure 3-6. Slope-Width and Depth-Width Solutions Calculated by SAM for R7S3

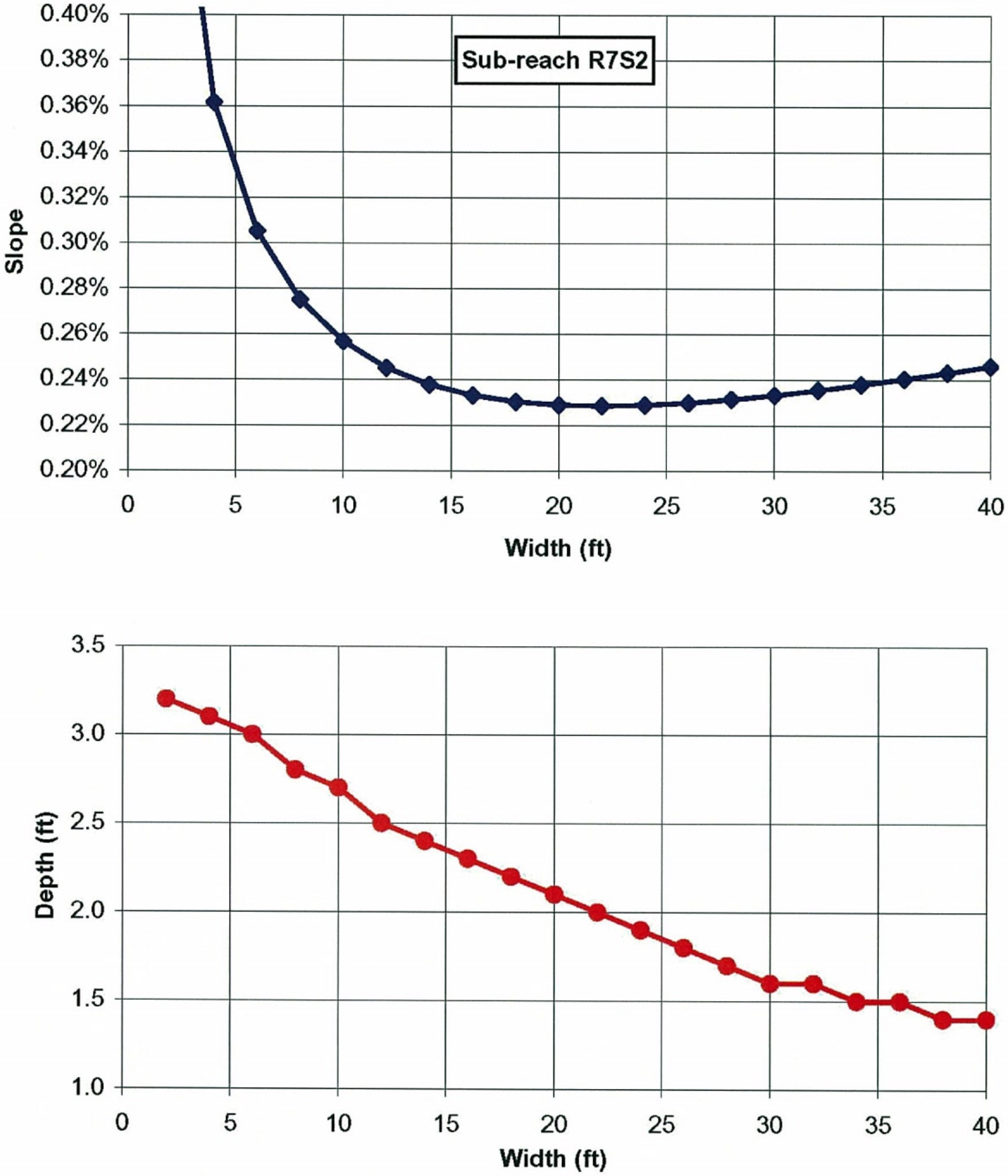


Figure 3-7. Slope-Width and Depth-Width Solutions Calculated by SAM for R7S2

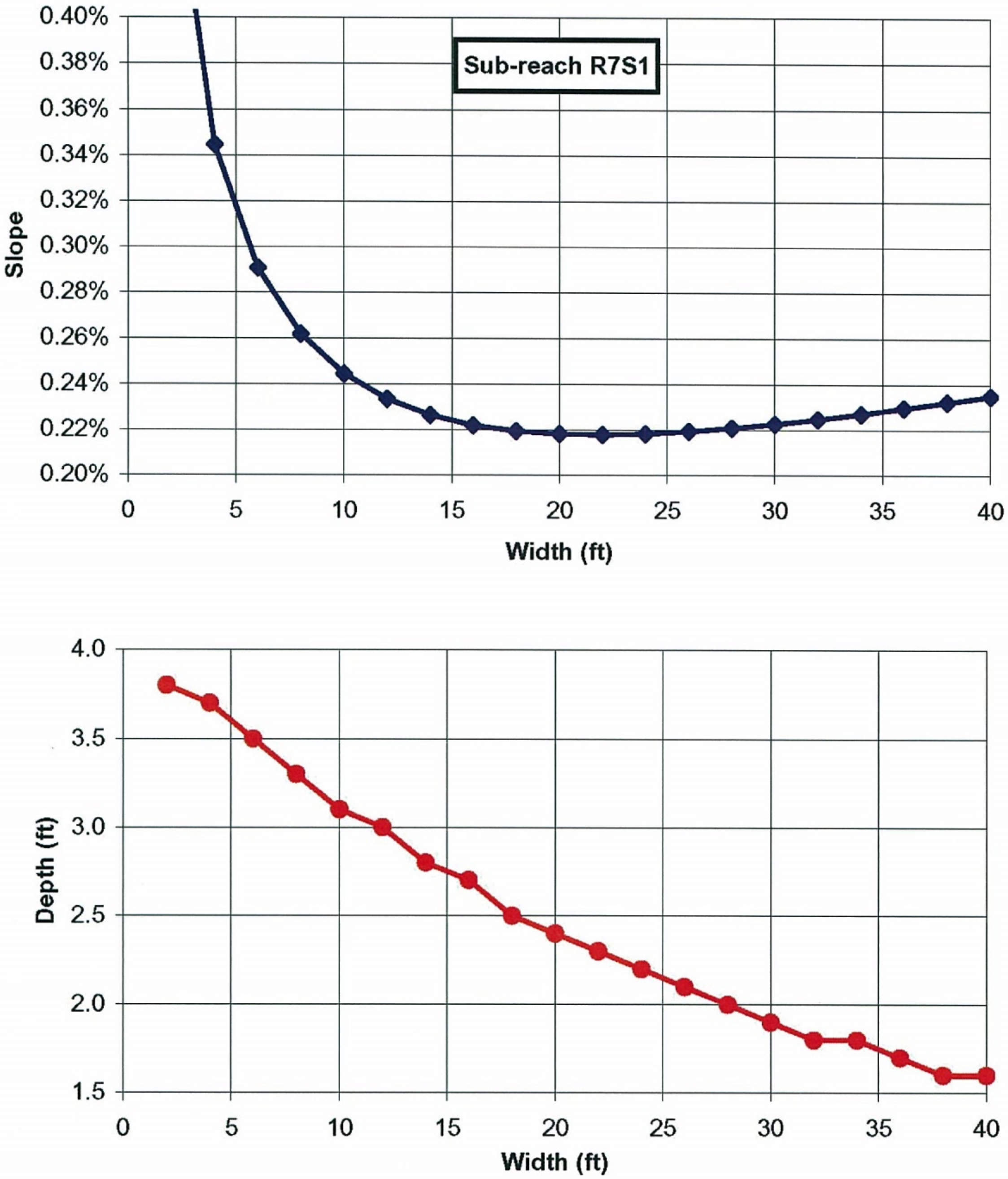


Figure 3-8. Slope-Width and Depth-Width Solutions Calculated by SAM for R7S1

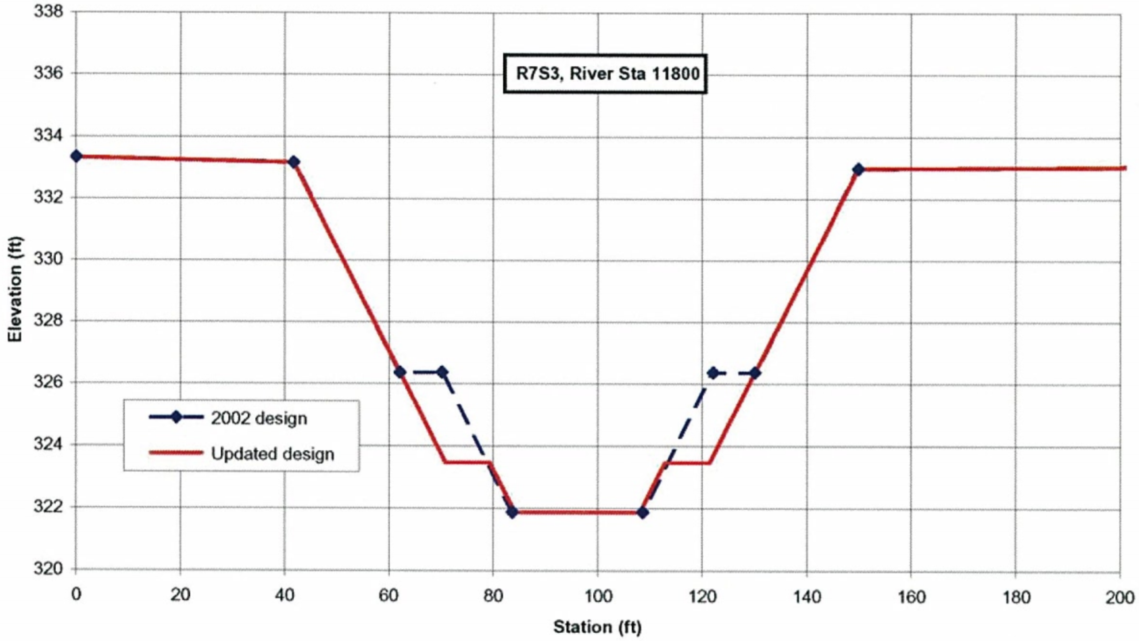


Figure 3-9. Updated Channel Design versus USACE-SFD (2002) Design for R7S3

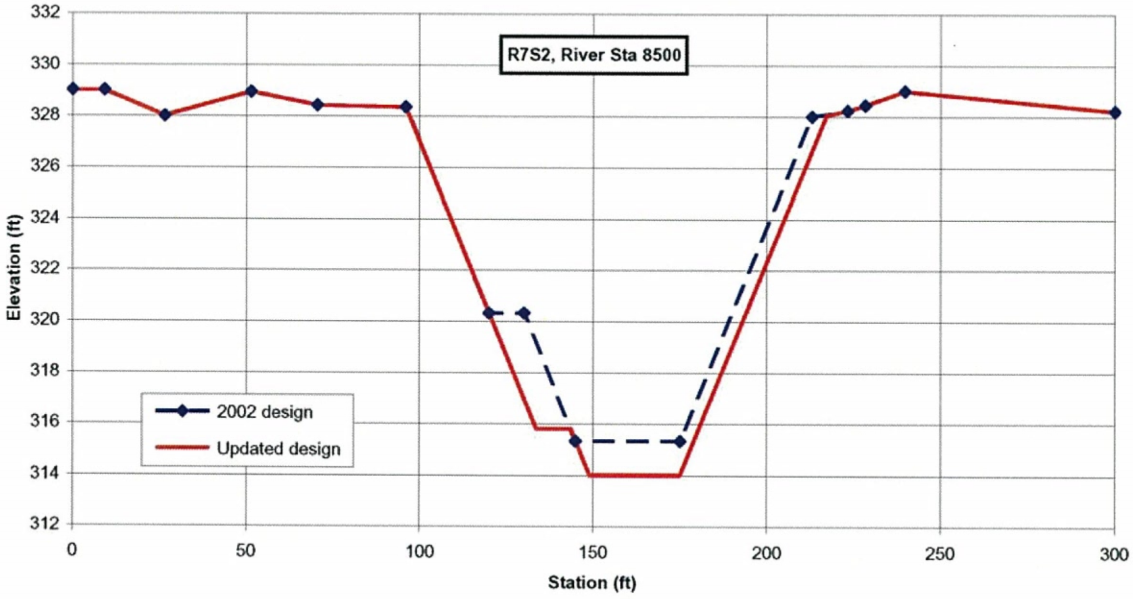


Figure 3-10. Updated Channel Design versus USACE-SFD (2002) Design for R7S2

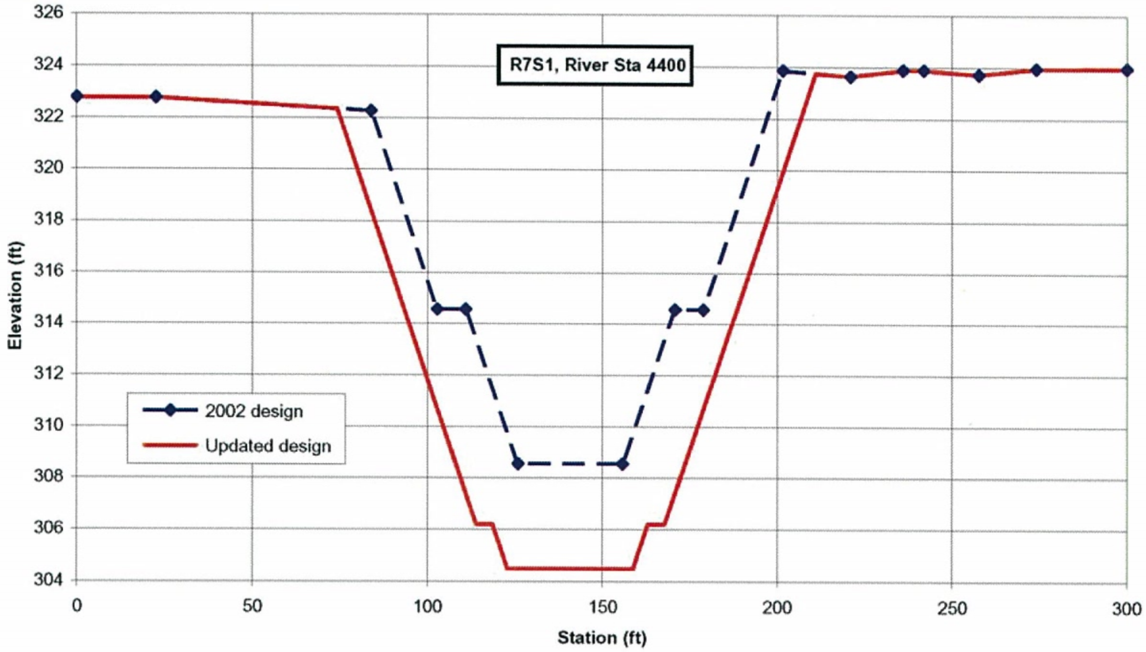


Figure 3-11. Updated Channel Design Versus USACE-SFD (2002) Design for R7S1

4.0 DIVERSION CHANNEL INITIAL AND LONG TERM CONDITIONS

As discussed in Section 3.0, the sediment budget in the diversion channel was analyzed using the mean sediment grain size distribution averaged over all the derived sediment distributions in Reach 7 and Reach 8. This method assumes that the bed material gradation in the diversion channel is already developed and is similar to the upstream reaches. It is a reasonable assumption for estimating the long term conditions of the diversion channel, but may not be applicable for the initial conditions. As a result, the sediment budget presented in Section 3.0 should only be considered as the long term sediment transport conditions. The initial sediment budget of the diversion channel was therefore calculated in this section, based on the initial bed gradation condition within the diversion channel.

As defined in the preliminary (USACE-SFD, 2002) study, Reach 7A is the channel segment between the La Crosse Street crossing and the confluence of the diversion channel to the main branch of Llagas Creek. It includes the diversion channel and a major portion of R7S2. As a result, the initial bed gradation of the diversion channel can be represented by the mean sediment grain size distribution derived from the URS sediment samples that were collected in Reach 7A only. This representative bed gradation is shown in Figure 3-5. The representative bed gradation for the long term conditions (averaged over Reaches 7A, 7B and 8) is also presented in the same figure. The initial bed gradation of the diversion channel (Reach 7A only) is very close to the long term conditions. The median sediment grain size (D_{50}) is 1.7 millimeters for the long term conditions and 2.0 millimeters for the initial conditions.

The computed initial sediment yield of the diversion channel R7S1 and R7S2 is compared to the long term conditions, as summarized in **Table 4-1**. The initial sediment yield was calculated to be approximately 12 to 14 percent less than the long term conditions for the considered flood scenarios owing to the coarser initial bed material gradation.

The computed initial sediment budget for different flow scenarios is presented in **Table 4-2,** compared to the long term conditions. The initial trap efficiency of the diversion channel will be greater than that for the long term conditions owing to the slightly smaller sediment transport capacity for the initial conditions. The trap efficiency will range from 26 percent to 45 percent for the initial conditions, compared to 18 percent to 38 percent for the long term conditions. This suggests that sediment will deposit faster in the diversion channel immediately after the construction, while the deposition rate will gradually slow down as the bed gradation adjusts to the condition that is similar to the upstream reach. It should be noted that the longitudinal gradient may also increase as sedimentation occurs in the diversion channel. This will also result in a decreasing trap efficiency with time in the diversion channel until the long term conditions are reached.

Table 4-1. Diversion Channel Initial and Long Term Sediment Yield for Option 4

Flow Event	Sub-reach	Water Yield (ac-ft)	Sediment Yield				
			Yang (tons)	Einstein (tons)	A-W (tons)	E-H (tons)	Mean (tons)
A	R7S2, Initial	3479	1507	296	3607	2200	1,372
	R7S2, Long-term	3,479	1,634	411	4,178	2,255	1,568
	R7S1, Initial	4273	1911	434	4749	2753	1,815
	R7S1, Long-term	4,273	2,070	595	5,496	2,819	2,090
B	R7S2, Initial	172	139	27	386	206	131
	R7S2, Long-term	172	150	37	448	211	151
	R7S1, Initial	211	179	36	480	256	168
	R7S1, Long-term	211	194	50	556	262	194
C	R7S2, Initial	966	1328	808	8536	2267	2,135
	R7S2, Long-term	966	1,428	1,069	9,940	2,321	2,436
	R7S1, Initial	1248	1963	1407	14248	3361	3,391
	R7S1, Long-term	1,248	2,111	1,834	16,597	3,441	3,856
D	R7S2, Initial	2022	3471	2628	29107	6356	6,409
	R7S2, Long-term	2,022	3,728	3,432	33,932	6,509	7,291
	R7S1, Initial	2681	5361	4748	52541	10019	10,759
	R7S1, Long-term	2,681	5,761	6,108	61,278	10,259	12,196

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

Table 4-2. Diversion Channel Initial and Long Term Sediment Budget for Option 4

Flow Event	Segment	Length (ft)	Water Yield (ac-ft)	Sediment (tons)		S-Y (tons)	$\frac{S-Y}{S}$
				Supply (S)	Yield (Y)		
A	Initial	5,040	4,273	2,921	1,815	1,106	38%
	Long-term	8,600	4,273	3,009	2,090	919	31%
B	Initial	5,040	211	305	168	137	45%
	Long-term	8,600	211	314	194	120	38%
C	Initial	5,040	1,248	4,748	3,391	1,357	29%
	Long-term	8,600	1,248	4,889	3,856	1,033	21%
D	Initial	5,040	2,681	14,528	10,759	3,769	26%
	Long-term	8,600	2,681	14,947	12,196	2,751	18%

Note: A: Average annual conditions B: Effective discharge
C: Ten-percent chance exceedance flood D: One-percent chance exceedance flood

5.0 FLOODPLAIN MAPPING

A flood mapping analysis was performed previously to assess the extent of the floodplain in the project area for various flood scenarios, based on the reconfigured channel design of Llagas Creek (NCI and **nhc**, 2007). The creek channel was initially designed in the preliminary study (USACE-SFD, 2002) and subsequently reconfigured in the 2007 study. After the preferred option of the revised diversion channel was selected in this study (as described in Section 3.2), a supplemental flood mapping analysis was required to re-evaluate the inundation stages for the same flood scenarios. It is noted that the newly modified channel configurations are applicable in Reaches 6 and 7 only.

Inundation stages and floodplain boundaries were determined for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year peak flows using the Corps HEC-RAS model for the project hydrologic conditions developed by the U.S. Army Corps of Engineers (USACE-SFD, 2006). The original HEC-RAS model was modified to incorporate the updated low-flow channel, the realigned diversion channel, and the overbank floodplain areas. Floodplain boundaries were extracted from the modified HEC-RAS model using GeoRAS and mapped in ArcMap.

5.1 Modification of HEC-RAS Model

The original HEC-RAS model was modified in this analysis to incorporate new low-flow channel designs developed in the previous study (NCI & **nhc**, 2007) as well as the updated modification in Reaches 6 and 7, as discussed in Section 3.2. The currently modified HEC-RAS model includes the updated low flow cross-sections for several sub-reaches to accommodate specific hydraulic needs or to comply with the site conditions. The invert elevations of cross sections in several sub-reaches (i.e. mainly Reaches 6 and 7) were modified based on a revised channel gradient for the purpose of mitigating the potential sedimentation problem for the originally proposed project conditions. The affected sub-reaches and the revised gradients are listed in Table 3-2. The geometries of three additional cross sections were also modified, based on the new design parameters. These cross sections were:

1. R7S3 from 137+00 to 99+00
2. R7S2 from 98+00 to 77+00
3. R7S1 from 76+00 to 21+00

Figure 3-9 through Figure 3-11 show examples of the updated channel designs for various sub-reaches. The modifications were made in accordance with Table 3-2 and the templates shown in Figure 3-9, Figure 3-10, and Figure 3-11. The return peak flows for the project conditions were set in accordance with Table 5 in the floodplain study report (USACE-SFD, 2006), which listed the derived peak flows of various flood events for the with-project conditions.

Structures that were associated with modified cross sections were updated by setting back levees and extending bridge decks appropriately. The sub-reach R7S1 was realigned downstream of river station 21+00. The channel between river stations 21+00 and 12+00 was removed and replaced by a realigned channel extending about 3,225 feet beginning at river station 21+00. The downstream end of the realigned channel was connected to sub-reach R6S2 at river station 693+00. The realigned channel had a gradient of 0.232%. New cross sections along that sub-reach were located at every 100-foot interval, based on the geometry of the cross section at 21+00 and the updated channel grade line. Because of the changes in the alignment of sub-reach R7S1, the West Little Llagas connection between Reach 7-8 and Reach 5-6 also changed. In the original 2002 USACE project model, those reaches were connected with the junction called "diversion". This junction was replaced by a new junction called "5-6" connecting the Reach 5-6, Reach 5-6 Lower and Reach 7-8. The HEC-RAS flow data file was also updated accordingly.

In addition to the incorporation of the most updated low-flow channel design, modifications of the original HEC-RAS model in the floodplain areas were also made to improve the floodplain mapping accuracy. The detailed discussion was presented in the 2007 study report (NCI & **nhc,** 2007). These modification measures include:

- Removal of the cross-sections that do not have associated spatial coordinates;
- Removal of the cross sections with spatial inconsistency between the flow centerline and cross-section bank stations; and
- Expansion of the floodplain areas to the original with-project cross-sections using the corresponding cross-sections from the HEC-RAS model for the existing conditions.

It is noted that the channel cross-sections in the original with-project HEC-RAS model are restricted to the main channel only and do not include the existing floodplains. As a consequence, the model underestimated the floodplain conveyance and resulted in overestimating the computed flood levels as well as underestimating the simulated inundation area. If there is no flood wall with an adequate height, then overestimating the computed flood levels would cause overestimate of the simulated inundation area. Incorporating floodplain conveyance by expanding the floodplain portion in the original with-project HEC-RAS model, the simulated water surface elevations were reduced by about 0.2 to 0.4 feet in the middle reach (Reaches 5, 6 and 14) and by around 1.5 feet in the lower reach (Reach 4). Concurrently, the computed inundation areas were expanded by a hundred feet in the middle reach and by over a thousand feet in the lower reach of the model.

5.2 Floodplain Mapping

Flooding in the study reach for project conditions was simulated using two HEC-RAS models:

- The "Project modified" (PM) model, which is based on the original with-project HEC-RAS model, incorporates the newly modified low-flow channel, and uses the revised hydrology; and

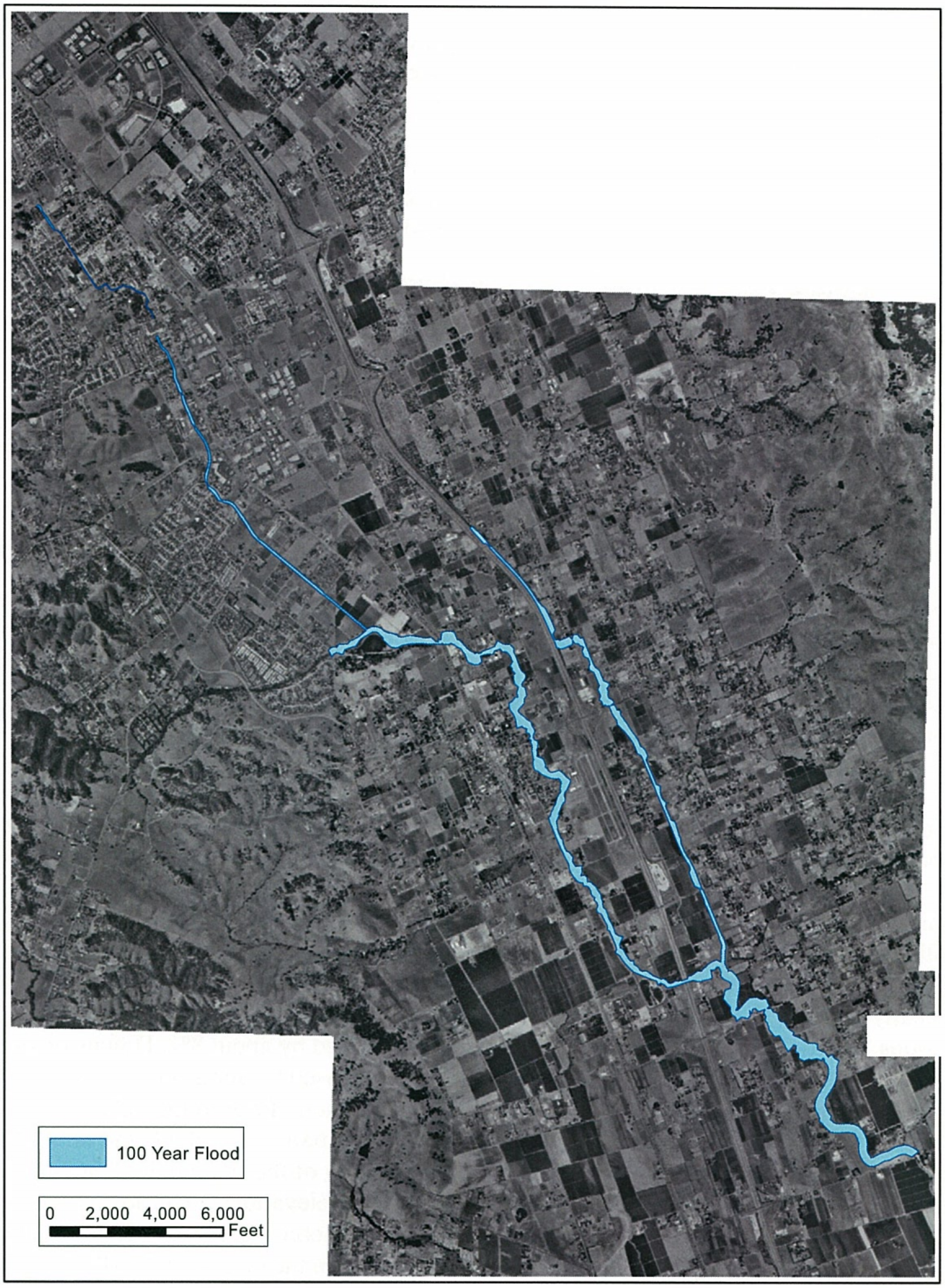
- The "Project modified extended" (PME) model, which is the above-mentioned modified model with cross-sections extended to incorporate floodplain areas (not included in the original HEC-RAS model).

In the absence of a digital elevation model representing the ground topography (which is required to create inundation depths) for the project conditions, only floodplain boundaries were extracted from the HEC-RAS models and plotted in ArcMap.

Figure 5-1 shows the 100-year inundation boundaries simulated using the PM model for the entire project area. **Figure 5-2**, **Figure 5-3** and **Figure 5-4,** show the 100-year inundation areas simulated using the PM model for the upper, middle, and lower project reaches, respectively. It is obvious from these figures that the use of the PM model (which does not include floodplain areas) is inadequate for mapping high flows which exceed the capacity of the main channel. The simulated 100-year inundation area in Figure 5-1 and Figure 5-2 is limited by the spatial extent of the model cross-sections rather than by physical floodplain boundaries.

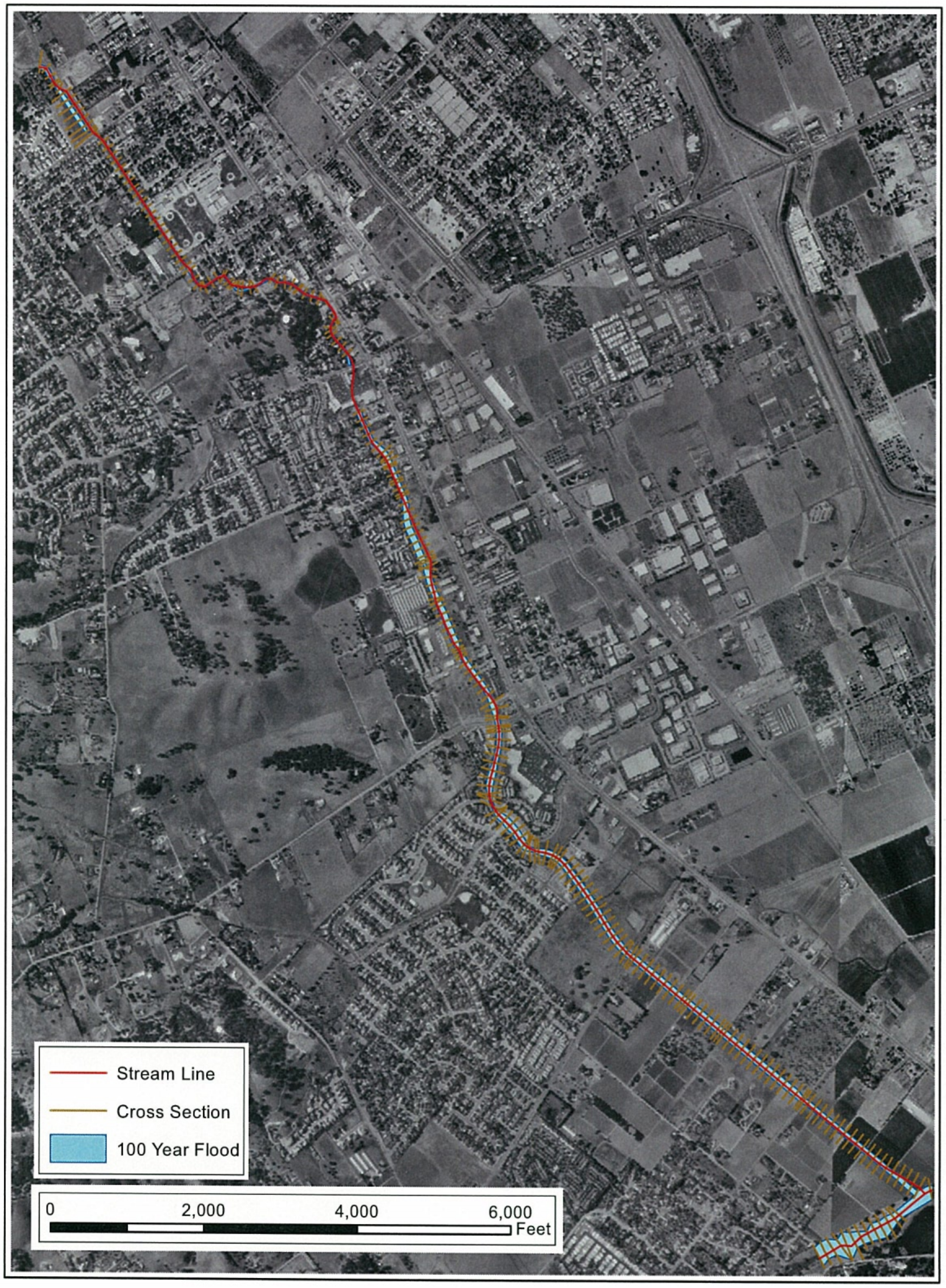
Figure 5-5 shows the 100-year inundation boundaries simulated using the PME model for the entire project area. **Figure 5-6**, **Figure 5-7**, and **Figure 5-8** respectively show the 100-year inundation areas simulated using the PME model for the upper, middle, and lower project reaches. In these figures, inundation area is not limited by the model boundaries, although the level of detail is lower than that produced by the PM model due to fewer cross-sections included in the PME model.

A comparison of the floodplain mapping areas for the 2007 and latest (2008) modified HEC-RAS models was performed for the specified return flood conditions. In general, water surface elevations are lower in the 2008 modified HEC-RAS model than in the 2007 modified model. The flood maps developed from each of the models are very similar; however, several differences are noted. One significant change was due to the realignment of the diversion channel in the downstream segment of Reach 7-8, shown in **Figure 5-9**. In Reach 7-8 between cross sections 76+00 and 21+00, flood depths are typically lower in the 2008 modified model due to the channel re-grading. Between sections 76+00 and 32+00, the average top width is reduced by about 8%. Downstream, between sections 30+00 and 21+00, the top width is increased by about 21% since the re-graded channel is wider and the invert is lower in this section. Prior to the 2008 modification, the water surface elevation of cross sections 686+00, 688+00 and 689+00 was just over the top of bank, resulting in shallow flooding of the floodplain. With the current (2008) modifications, slightly lower water surface elevations are produced, as illustrated in **Figure 5-10**. This reduction, however, is not considered significant because of the very shallow depths of flooding that was simulated in the original HEC-RAS model.



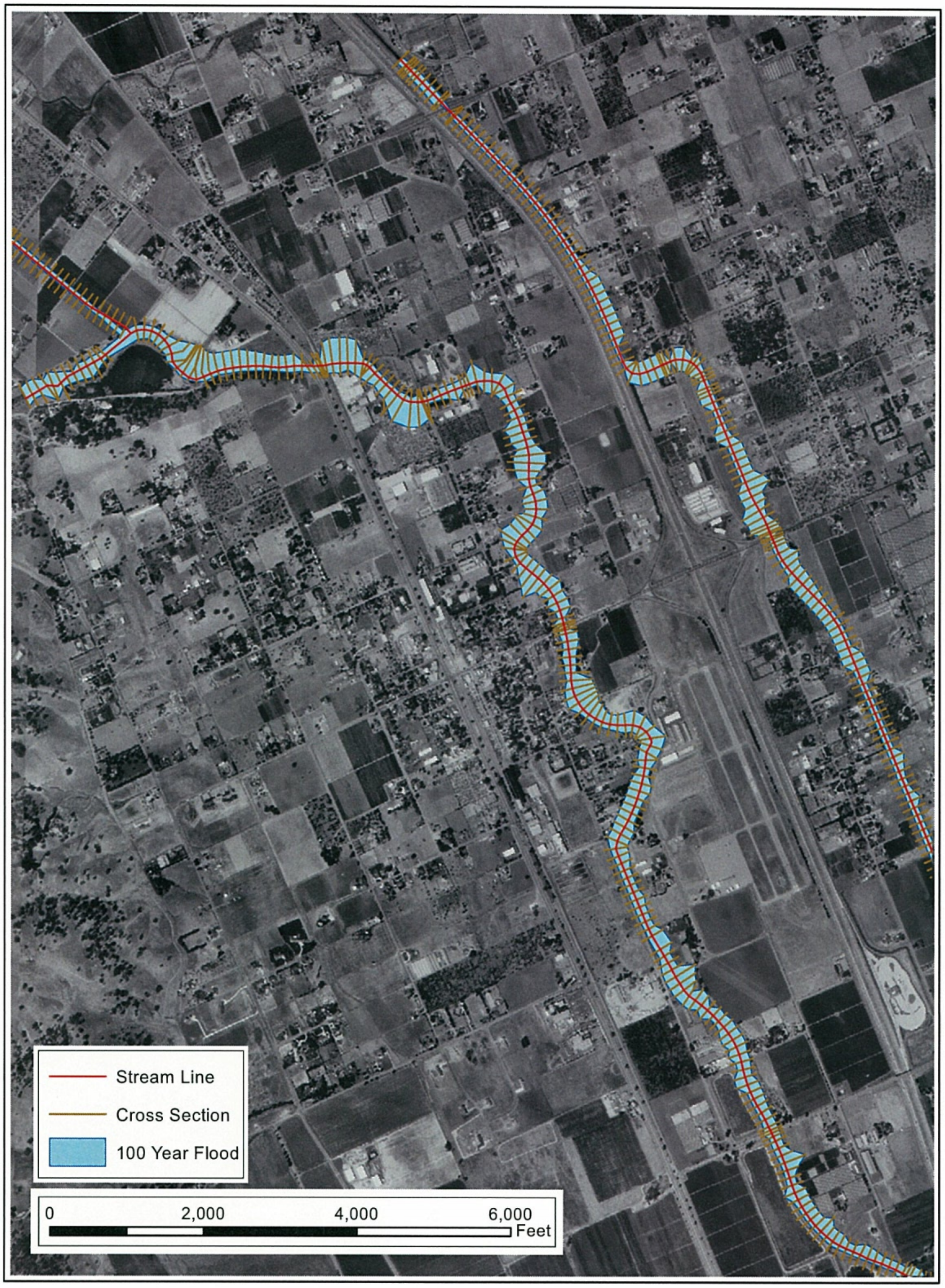
Note: Model cross-sections are as provided by USACE, i.e. not including existing floodplain areas.

Figure 5-1. 100-Year PM Inundation Boundaries for the Entire Reach



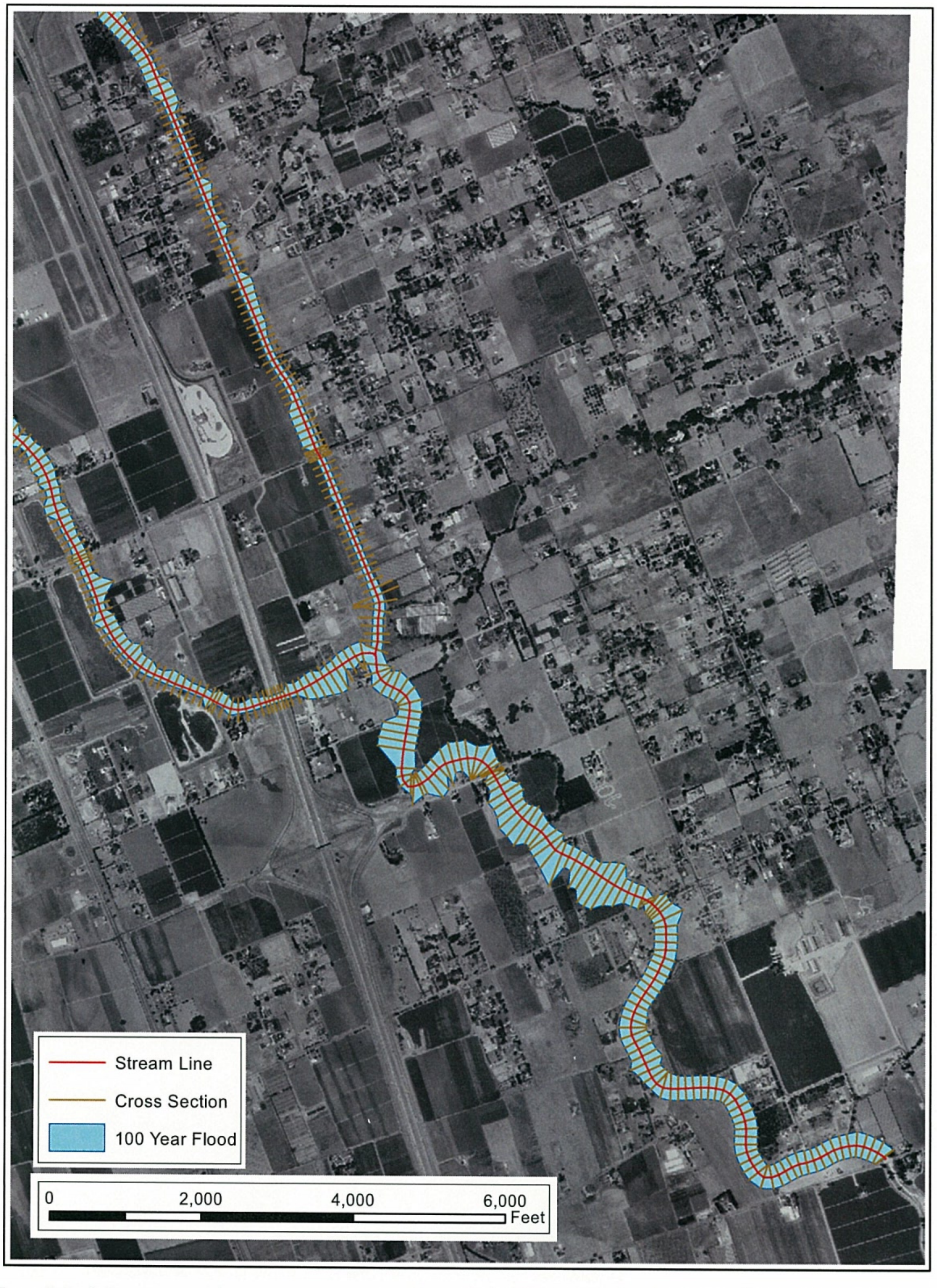
Note: Model cross-sections are as provided by USACE, i.e. not including existing floodplain areas.

Figure 5-2. 100-Year PM Inundation Boundaries for the Upper Reach



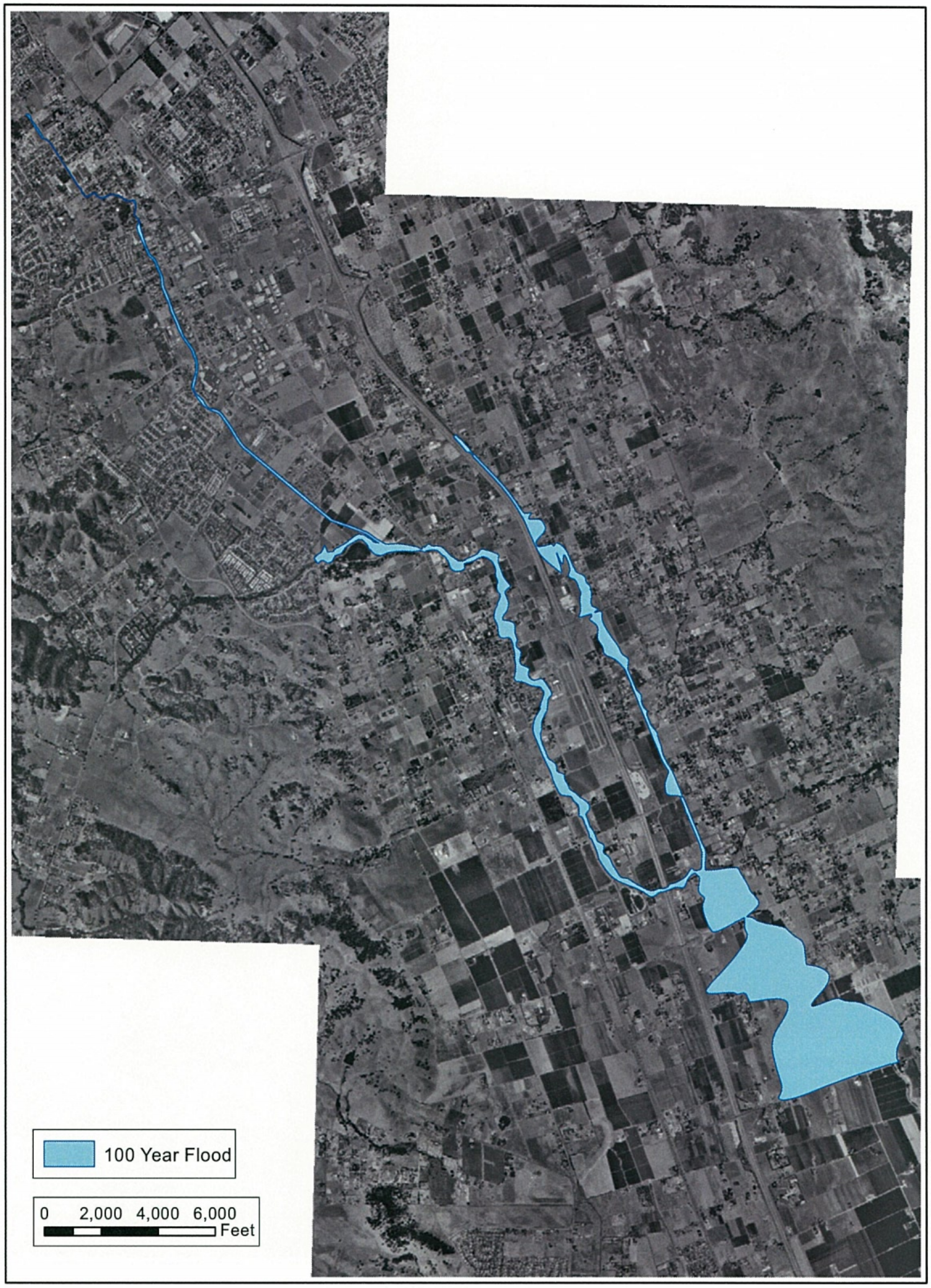
Note: Model cross-sections are as provided by USACE, i.e. not including existing floodplain areas.

Figure 5-3. 100-Year PM Inundation Boundaries for the Middle Reach



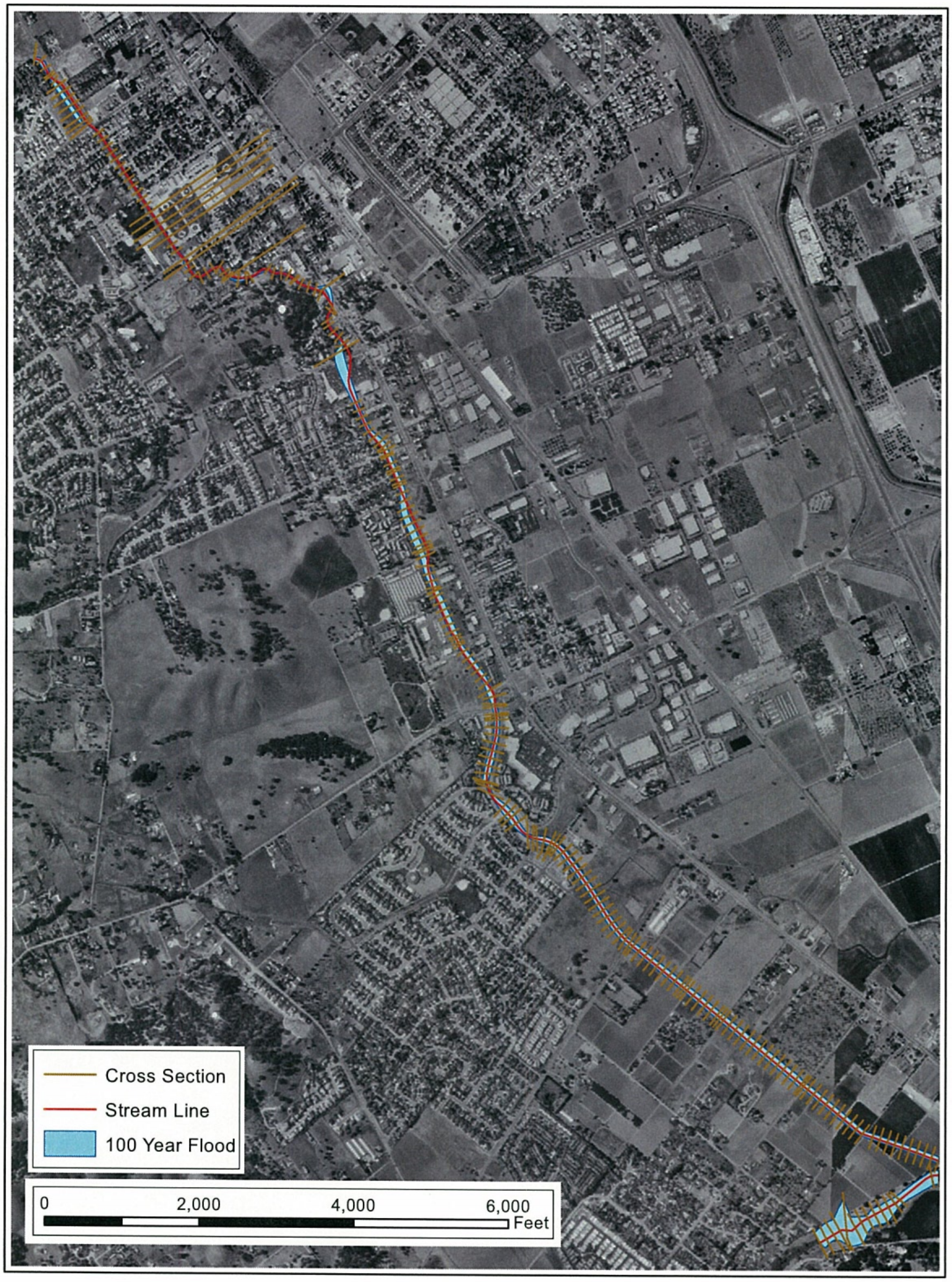
Note: Model cross-sections are as provided by USACE, i.e. not including existing floodplain areas.

Figure 5-4. 100-Year PM Inundation Boundaries for the Lower Reach



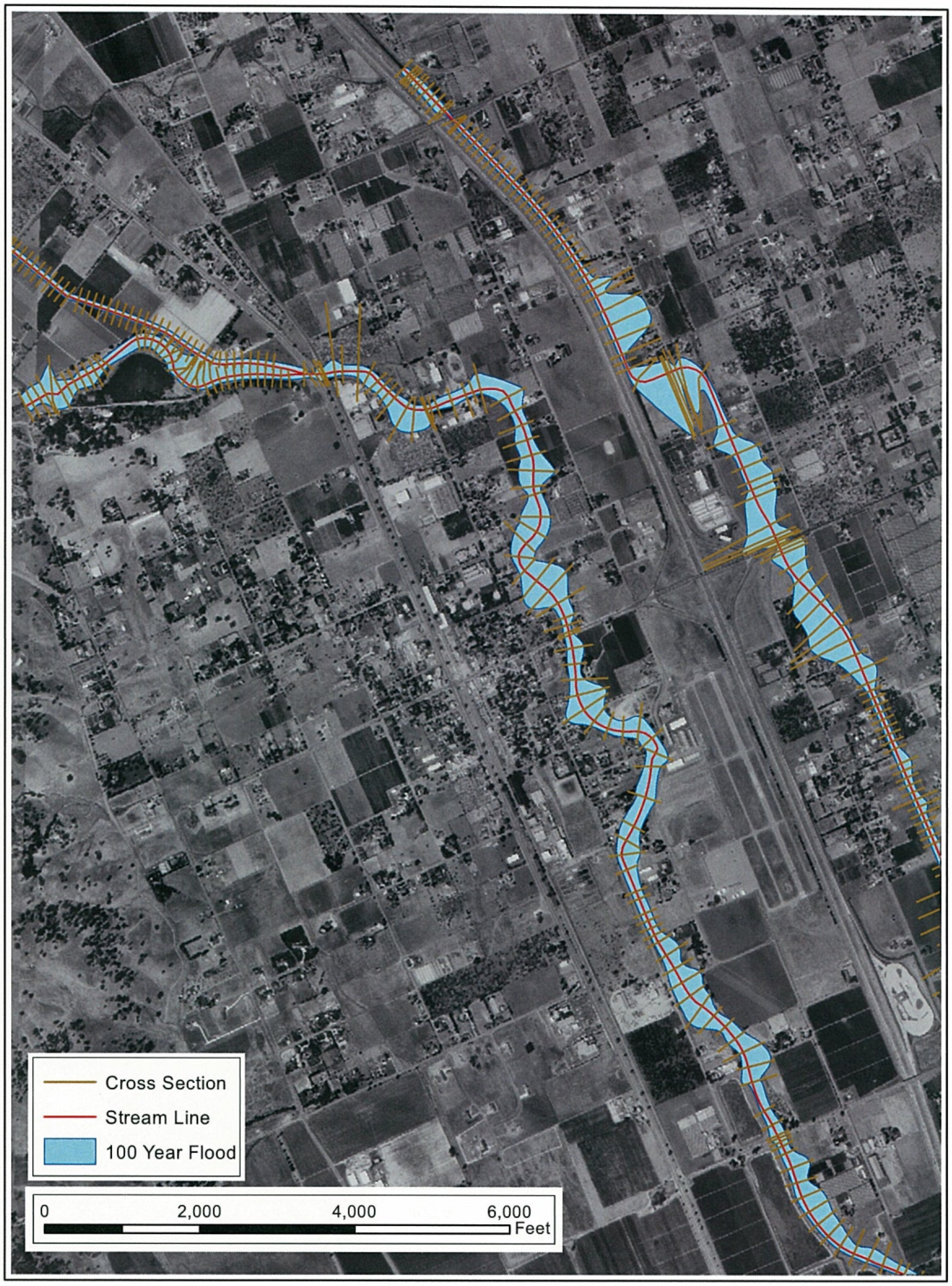
Note: Model cross-sections are extended laterally and include existing floodplain areas.

Figure 5-5. 100-Year PME Inundation Boundaries for the Entire Reach



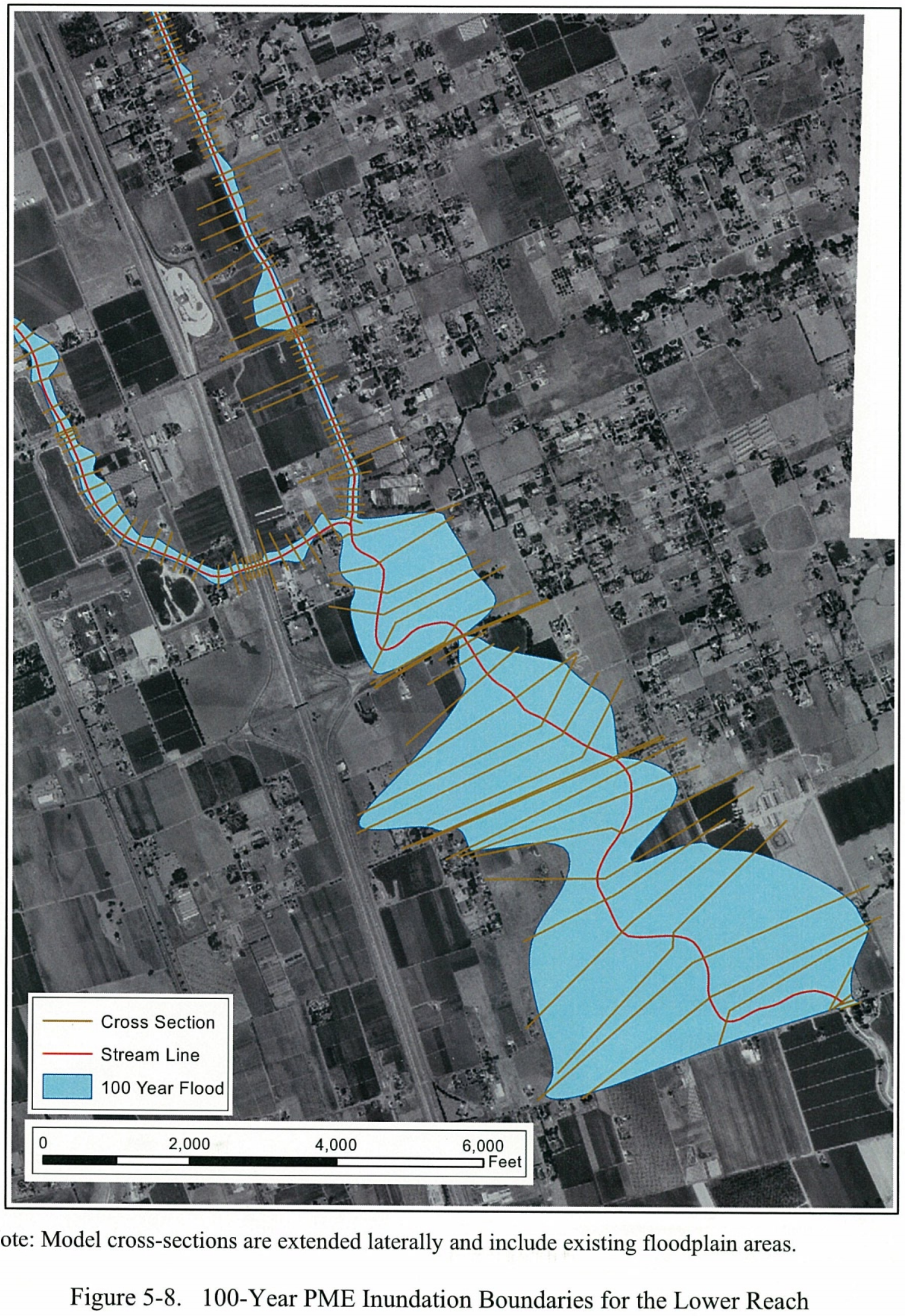
Note: Model cross-sections are extended laterally and include existing floodplain areas.

Figure 5-6. 100-Year PME Inundation Boundaries for the Upper Reach



Note: Model cross-sections are extended laterally and include existing floodplain areas.

Figure 5-7. 100-Year PME Inundation Boundaries for the Middle Reach



Note: Model cross-sections are extended laterally and include existing floodplain areas.

Figure 5-8. 100-Year PME Inundation Boundaries for the Lower Reach

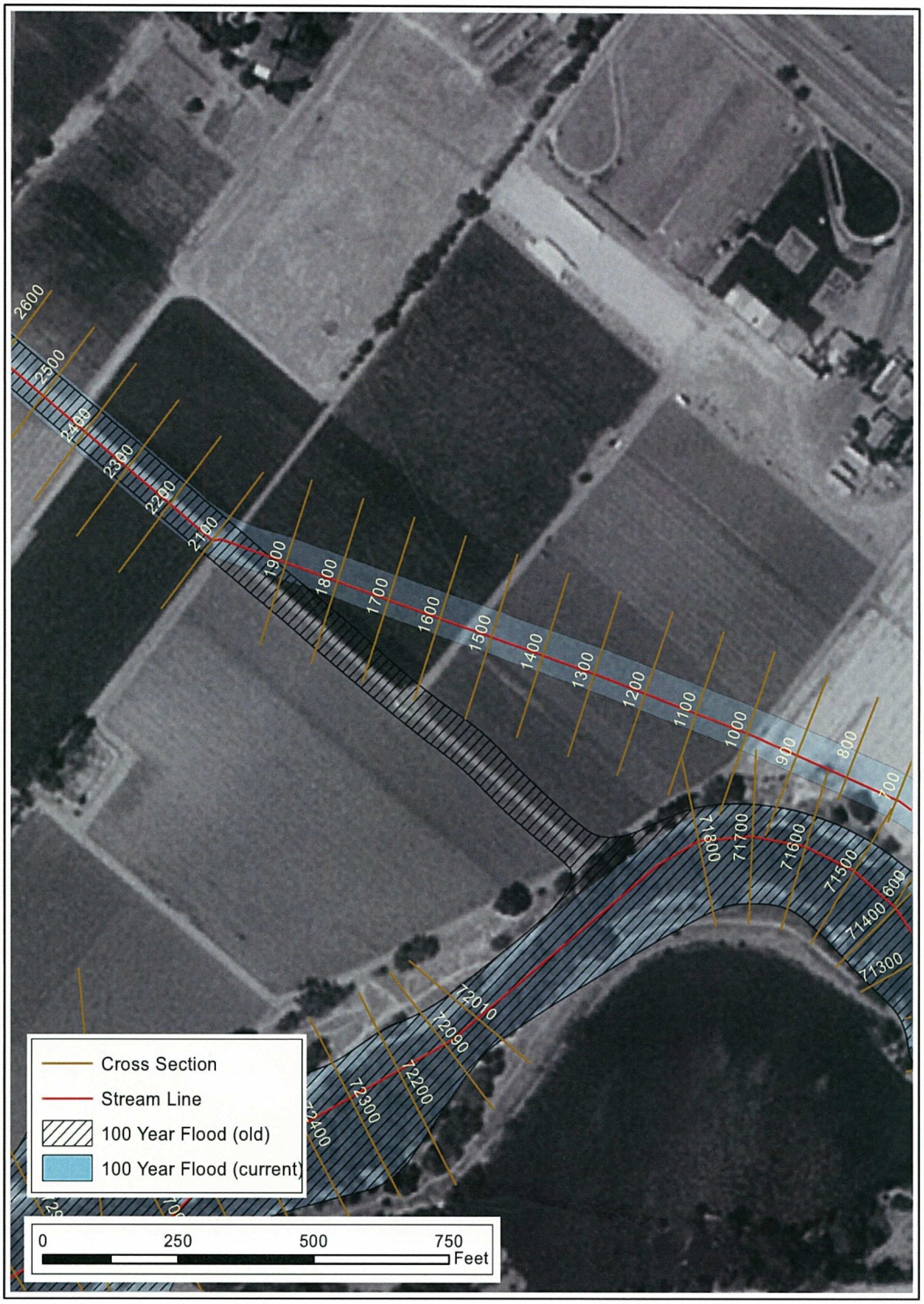


Figure 5-9. Comparison of Flood Maps for Realigned Diversion Channel between the 2007 and 2008 Modified Conditions

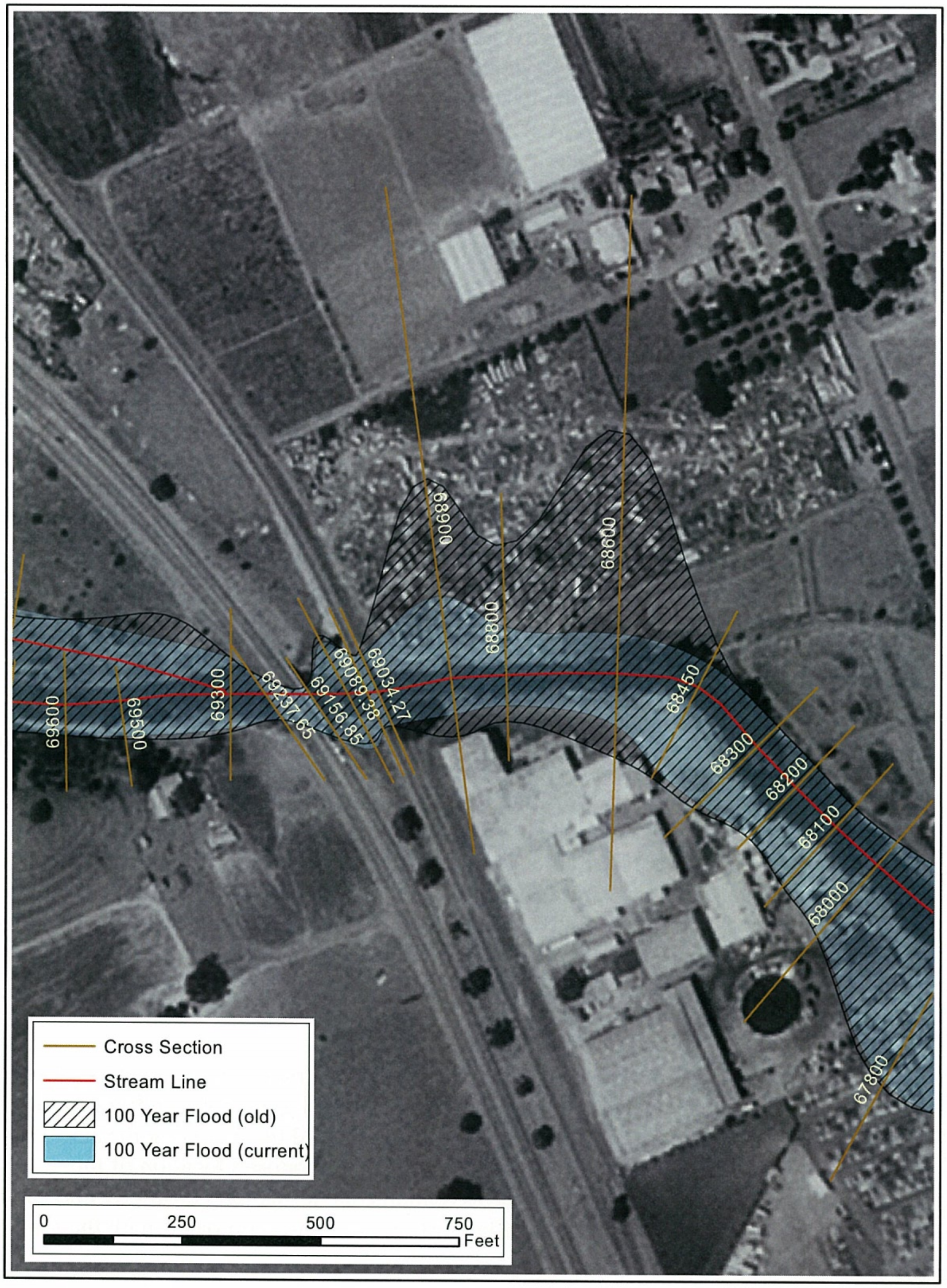


Figure 5-10. Comparison of Flood Maps in Areas of Local Flooding between 2007 and 2008 Modified Conditions

6.0 SUMMARY

This supplementary analysis was conducted to identify a preferred design option for the diversion channel. The design shall improve the performance and sediment transport capabilities of the diversion channel, alleviate the sedimentation problem as identified in the previous study (NCI & **nhc**, 2007), minimize the potential adverse impact to adjacent Llagas Creek, and move the sediment through the system in an acceptable manner.

Six new options, including the two proposed in the previous study (NCI & **nhc**, 2007), were initially formulated for the diversion channel design. Three options were selected for further in-depth sediment budget analysis after the initial screening. These three design options include (1) an option with a sedimentation basin built at a location upstream of the diversion channel, (2) a re-grading option, and (3) a re-alignment option. The sedimentation basin option will reduce the sediment supply to the diversion channel. The other two options will increase the sediment transport capability of the diversion channel.

The sediment budget computed for each selected option was compared to the previous design (NCI & **nhc**, 2007). The sediment yield was computed based on the mean bed gradation, which was calculated by averaging the soil samples (URS, 2006) collected within Reach 7 and Reach 8. The channel geometry of the diversion channel and adjacent reaches was updated for the re-grading and re-alignment options before conducting the sediment budget analysis.

The three mitigation options will all result in a much smaller trap efficiency in the diversion channel compared to the original design (NCI & **nhc**, 2007). In other words, the mitigation options will all significantly alleviate the sedimentation problem in the diversion channel associated with the original design. Different mitigation options will have different impacts to the hydraulic, sedimentation and environmental conditions in the adjacent reaches of Llagas Creek, particularly in Reach 6 which is located immediately downstream of the diversion channel. The re-alignment option was selected as the preferred design alternative after evaluating the sediment budget, potential impacts to creek stability, potential impacts to the environmentally sensitive area, and flood capacity of the upstream reach.

The initial sediment transport conditions of the diversion channel were evaluated for the selected preferred design option using the initial bed gradation conditions of the diversion channel. The initial bed gradation was determined by averaging the soil samples (URS, 2006) collected within the vicinity of the proposed location of the diversion channel (Reach 7A). The initial diversion channel will have slightly coarser bed gradation, lower sediment yield, and larger trap efficiency compared with the long term conditions.

The modifications made to the HEC-RAS geometry for the preferred design option does not result in substantial changes in the water surface profiles except at cross sections where the invert grade drops. This preferred design suggests a gradual invert drop along a part of Reach 7 and Reach 6, with a maximum drop of approximately 6 feet. Water surface elevations in these reaches will also drop accordingly.

7.0 REFERENCES

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